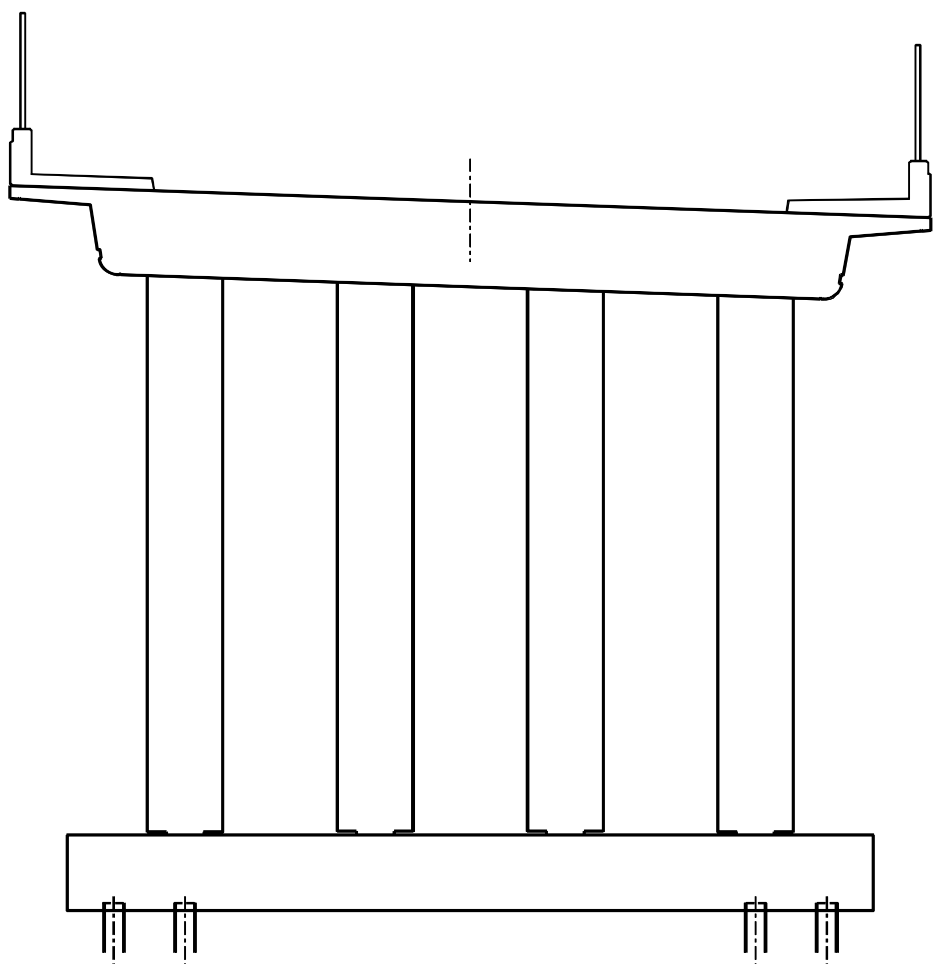
STRUCTSOURCE®

LRFD DESIGN OF CALIFORNIA BRIDGES

BENT DESIGN



Bob Matthews

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Note: These course notes were prepared based on AASHTO LRFD Bridge Design Specifications, 3rd edition, with 2005 and 2006 Interim Revisions, as amended by Caltrans, v3.06.01. Reference to Caltrans Seismic Design Criteria is for version 1.4.

Requirements based on Caltrans Amendments to AASHTO LRFD and Caltrans Seismic Design Criteria are highlighted for clarification.

SHEET 1-1 OF

JOB TITLE BENT DESIGN | ORIGINATOR Bob Matthews | DATE 10/19/2007

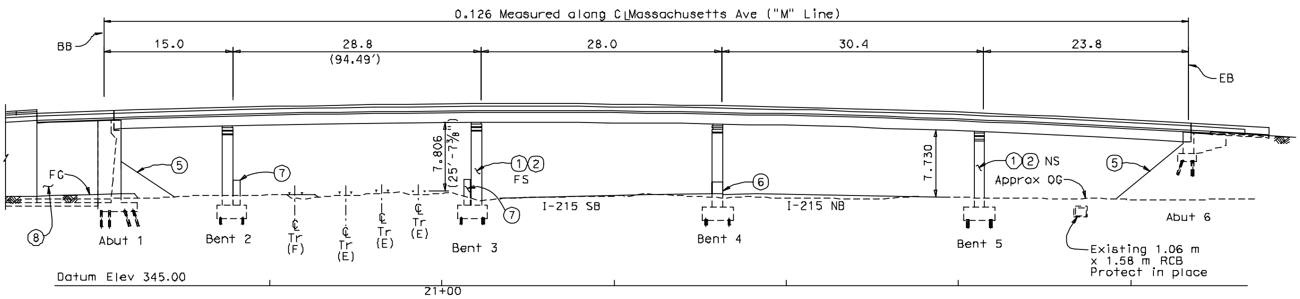
JOB No. | CALCULATION No. | REVIEWER | DATE

SECTION 1.0 BENT CONFIGURATION

Note: The bridge configuration was taken from Massachusetts Avenue Overcrossing of Interstate 215 in San Bernardino California.

<u>1.1 Bridge configuration and bent locations</u>

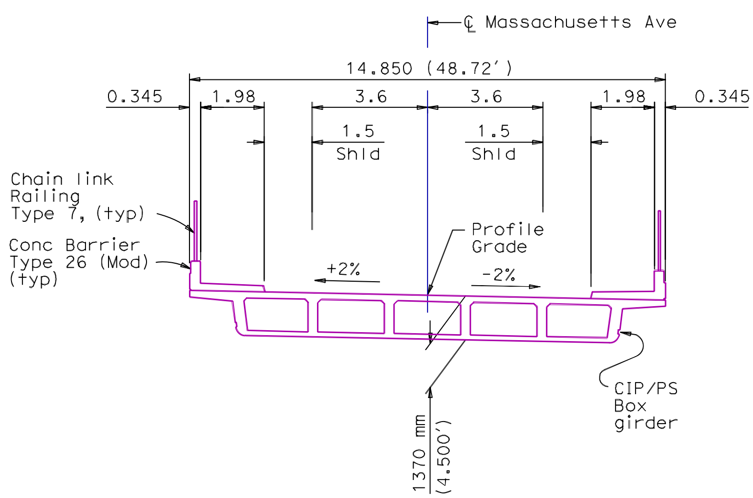
The bents were placed as shown below to span over railroad tracks and roadway.



SPAN	LENGTH
1	15.0 m (49.21')
2	28.8 m (94.49')
3	28.0 m (91.86')
4	30.4 m (99.74')
5	23.8 m (78.08')

ABUT/BENT	*HEIGHT
1	9.25 m (30.35')
2	9.78 m (32.09')
3	10.98 m (36.02')
4	10.73 m (35.20')
5	9.76 m (32.02')
6	4.74 m (15.55')

*Height is profile grade elevation minus top of footing elevation.



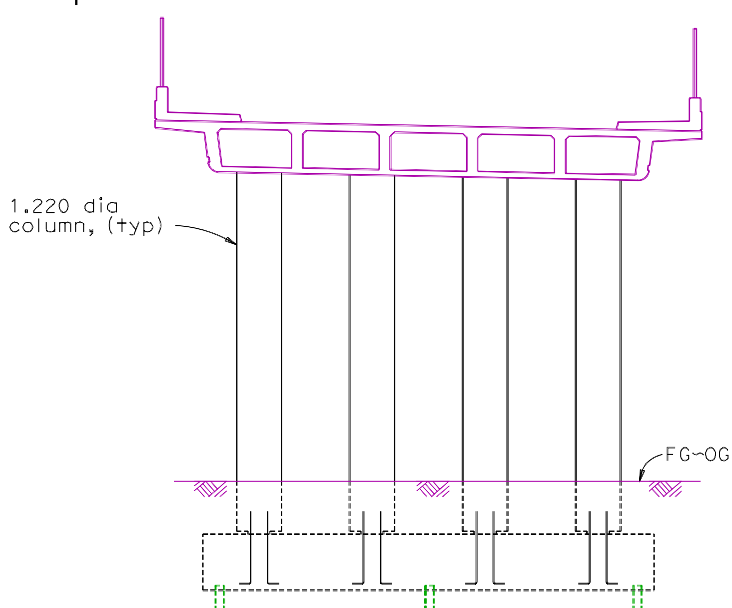
SUPERSTRUCTURE SECTION

1.2 Configure bent

The following considerations are used to configure the bent.

ITEM	CONSIDERATIONS
Bent cap size	• Match superstructure depth except with outriggers for better aesthetics and constructability. • Depth to develop column reinforcement and/or width to accommodate column hooks. • Width as needed for rebar placement, especially at column interface. • Size as needed to carry loads. Minimum of 2' wider than column for seismic/joint shear reinforcement (SDC 7.4.2.1)
Column size and location	• Avoid interference with superstructure prestressing tendons • Number and size as needed to carry loads • Depth less than superstructure for seismic (SDC 7.6) • Avoid knee joints at exterior columns for seismic (SDC 7.4.3) • Limit column slenderness for seismic (SDC 4.2) • Aesthetic considerations
Foundation type and location	• Geotechnical recommendations (spread footing, pile footing, etc.) • Seismic performance requirements (e.g. repairable damage) • Footing under roadway structural section (~2') • Avoid utilities (e.g. drain inlets, irrigation lines, etc.) • Lower footing as needed to balance stiffness for seismic (SDC 7.1)

The bent configuration is shown below. The columns were pinned at the base to reduce the foundation loads. 625 kN (70 ton) HP360x132 (HP14x89) piles were selected based on geotechnical recommendations. Three columns would be more typical for this section, but four were required to satisfy seismic requirements.



BENT ELEVATION

SHEET 1-3 OF

JOB TITLE **BENT DESIGN** ORIGINATOR Bob Matthews DATE 10/19/2007

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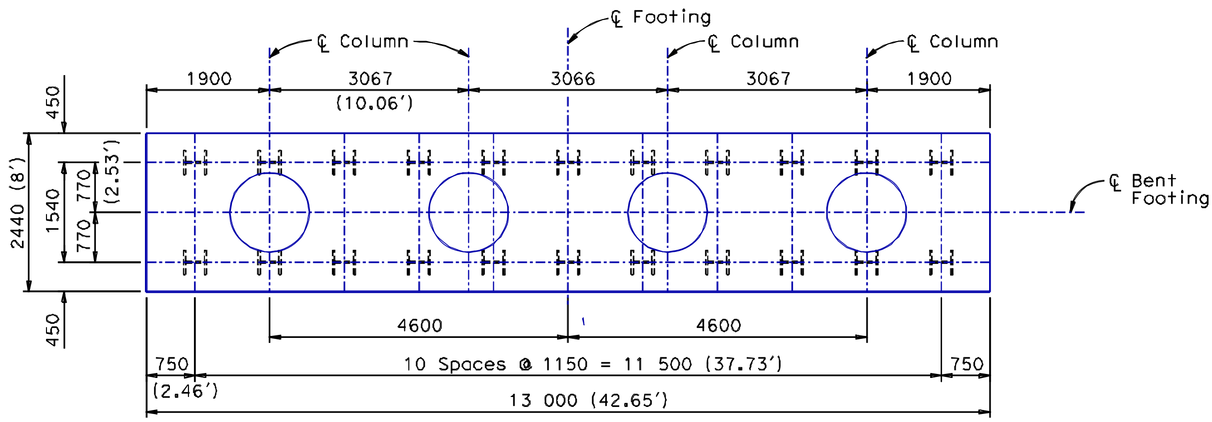
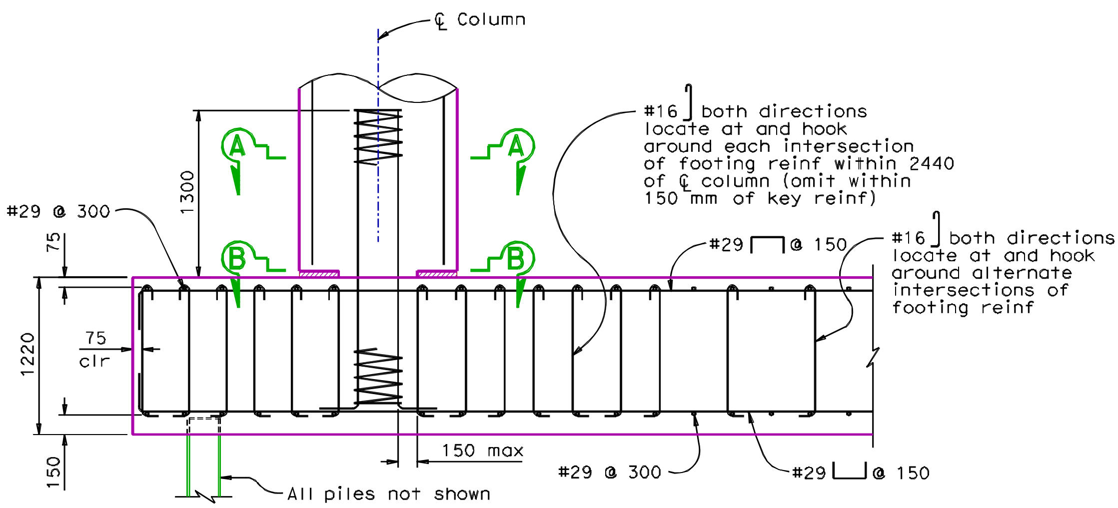
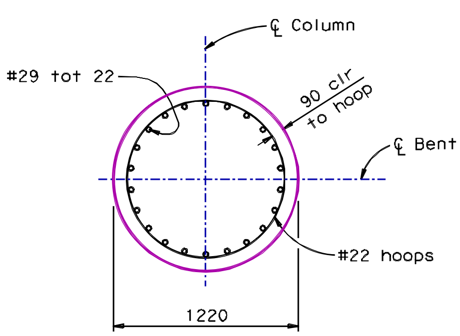


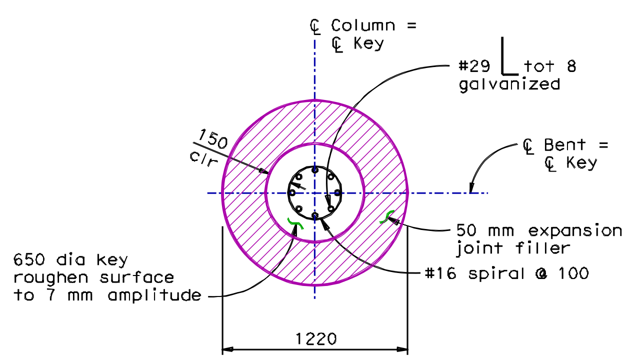
FOOTING PLAN



FOOTING DETAIL



SECTION A-A



SECTION B-B

SHEET 1-4 OF

JOB TITLE **BENT DESIGN** ORIGINATOR Bob Matthews DATE 10/19/2007

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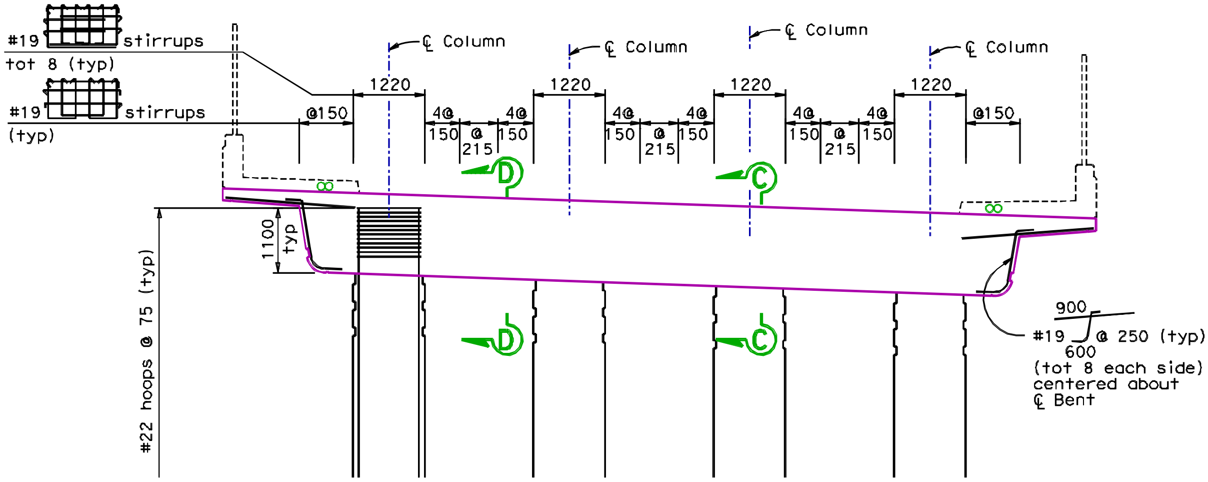
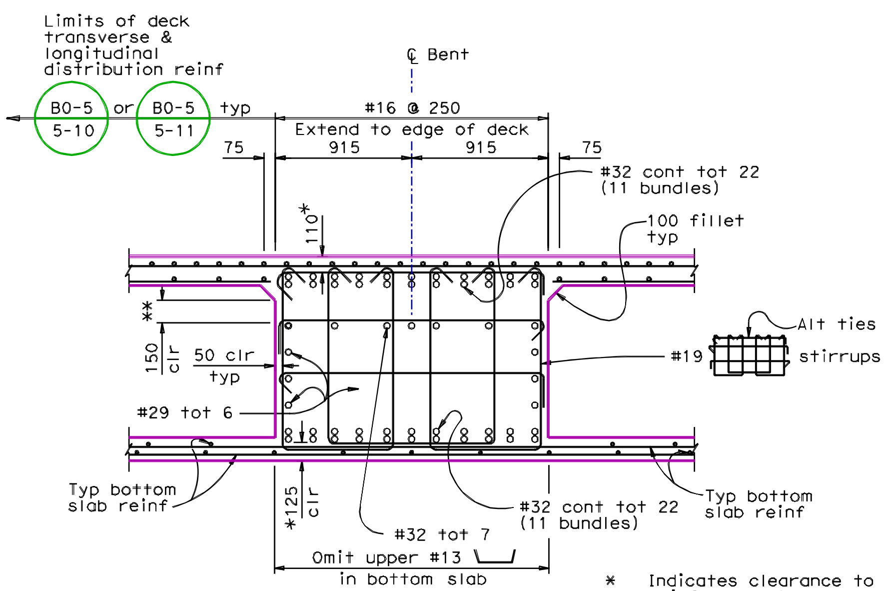


ELEVATION



SECTION D-D

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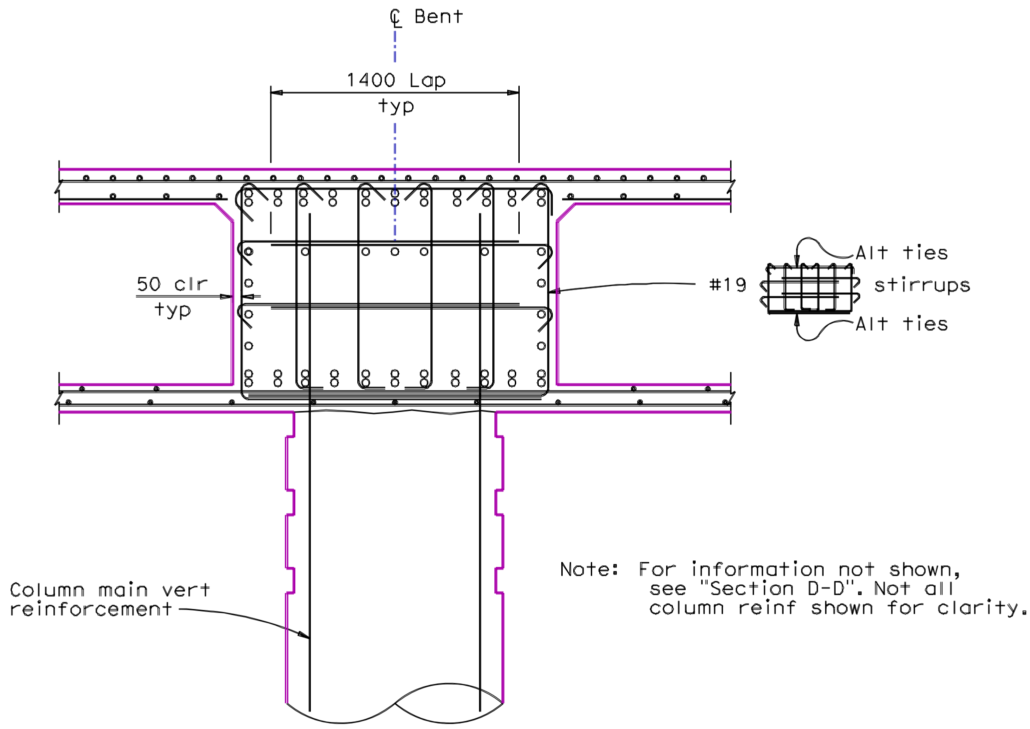
* Indicates clearance to main cap reinforcement

** Indicates clearance to #32 tot 7. Reinforcement may be lowered to clear prestress ducts; however, this dimension shall not exceed 200 unless approved by the Engineer.



SECTION C-C

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SECTION 2.0 BENT LOADS

Loads will be determined at Bent 5.

2.1 PERMANENT LOADS (LRFD 3.5 & Caltrans Amendment 3.4.1)

- Caltrans considers secondary prestress forces, creep and shrinkage as permanent loads and gives them different load factors from AASHTO LRFD.

2.1.1 Dead load constant (DC):

- The longitudinal load at the bent due to the superstructure dead load (constant) should be determined using a global model to account for moment distribution effects with the superstructure. CONBOX was used to determine the forces shown below.

Load	Longitudinal Force (kips)	Vertical Force (kips)	Bending Moment (k-ft)
Superstructure	-20.65	1080.66	-568.50
Barrier	-5.08	266.42	-139.92
Total	-25.73	1347.08	-708.42
Total per girder	-4.3	225	-118

Bent cap (filled void area) = 3.35 x 32.33 x 6 x 0.15 = 97.5 kips
Columns = 3.1416 x $(4)^2$ / 4 x 27.5 x 4 x 0.15 = 207.3 kips
Footing = 42.65 x 8 x 4 x 0.15 = 204.7 kips
Soil on footing = 2 x 0.12 = 0.24 ksf
= [42.65 x 8 – 12.57 x 4] x 0.24 = 69.8 kips

2.1.2 Dead load varying (DW):

- The dead load varying includes future AC overlay.
- The longitudinal load at the bent due to the superstructure dead load (varying) should be determined using a global model to account for moment distribution effects with the superstructure. CONBOX was used to determine the forces shown below.

Load	Longitudinal Force (kips)	Vertical Force (kips)	Bending Moment (k-ft)
AC	-3.21	167.99	-88.23
Total per girder	-0.5	28	-14.7

2.1.3 Secondary prestress forces (PS): (Caltrans Amendment 3.12.7)

- The longitudinal load at the bent due to secondary prestress force should be determined using a global model to account for moment distribution effects with the superstructure. CONBOX was used to determine the forces shown below.

Load	Longitudinal Force (kips)	Vertical Force (kips)	Bending Moment (k-ft)
Prestress	77.43	-20.58	2131.28
Total per girder	12.9	-3.4	355

2.1.4 Creep (CR) and Shrinkage (SH): (LRFD 3.12.4, 3.12.5)

- LRFD 5.9.5.3 provides approximate and refined methods to determine the time-dependent losses in the post-tensioning reinforcement.

- The effects of these losses are included in the CONBOX prestress (PS) force at the bent as shown in section 2.1.3 above. This was confirmed with the software developer.

- Shrinkage effects in the plane of the bent will be ignored since the bent is not that wide.

2.2 Live Load (LRFD 3.6)

- Caltrans Amendments include the Permit Vehicle (3.6.1.8 & 3.6.2), application of live load (3.6.1.3) and fatigue load (3.4.1 & 3.6.1.4). These provisions affect the superstructure analysis used to determine the live load reactions on the bent.

- Determining the exact live load distribution on a bent is impractical due to the load placement variations and modeling constraints.

- Live loading may be idealized based on typical California Department of Transportation practice. Refer to Caltrans Bridge Design Practice section 2, February 1994.

Assumptions:

1. Live loads for each lane are idealized as two concentrated loads 6' apart (LRFD 3.6.1.2.3 and Caltrans Bridge Design Practice section 2). The uniform lane load reaction is included with the truck load reaction and will not be modeled separately.

2. The concentrated loads are located within a 12' wide traffic lane, no less than 2' from the edge of the lane or 1' from the edge of an overhang barrier (LRFD 3.6.1.3.1).

3. At least four load cases need to be considered.

   a. HL-93 max axial with associated longitudinal moment
   b. HL-93 max longitudinal moment with associated axial
   c. Permit max axial with associated longitudinal moment
   d. Permit max longitudinal moment with associated axial

4. Additional HL-93 cases may need to be considered for multi-lane bridges using the Multiple Presence Factor from LRFD 3.6.1.1.2. You need to consider which load will give you the maximum positive and negative moments and shears in the bent cap, critical column axial-moment interaction loading and maximum load on the foundation.

Number of loaded lanes	Multiple presence factor
1	1.20 (1.0 for permit)
2	1.00
3	0.85
>3	0.65

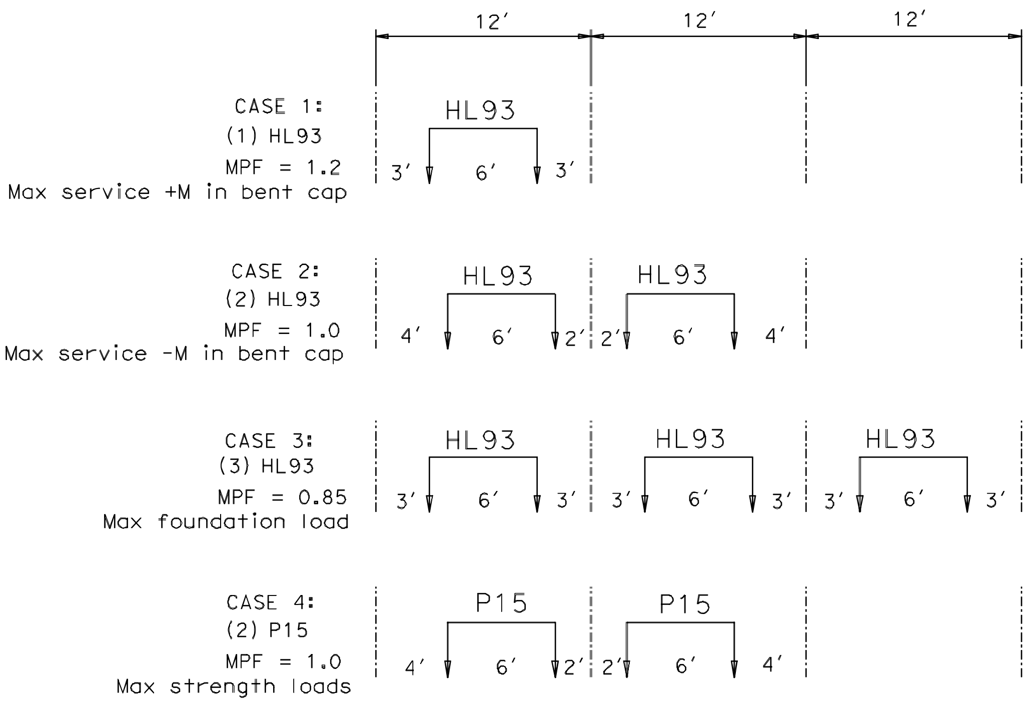
Effective live load lanes:

The bridge is striped for two lanes, but there is a possibility that it could be modified or restriped in the future for additional lanes.

This bridge includes (2) 3.6 m lanes, (2) 1.5 m shoulders and (2) 1.98 m sidewalks. LRFD commentary to 3.6.1.3.1 suggests including the sidewalks in the live load lane distribution if vehicles can mount them. This is consistent with Caltrans practice.

Width of potential driving surface = 14.16 m (46.46')
Number of effective lanes = 46.46 / 12 = 3.87 consider up to 3 lanes on the bridge.

Moving live load cases:



For the purposes of this example, analysis will only be performed for case 2 maximum axial and case 4 maximum axial.

- The lane reactions at the bent due to the superstructure live load should be determined using a global model to account for moment distribution effects with the superstructure. CONBOX was used to determine the forces shown below.

CASE	HORIZONTAL (Kips/lane)	VERTICAL (Kips/lane)	MOMENT (k-ft/lane)
HL93 max axial	-0.92	99.09	-25.34
HL93 max moment	-13.07	68.04	-359.76
P15 max axial	-5.41	290.67	-148.92
P15 max moment	-27.56	158.04	-759.05

The horizontal and moment loads will be applied at each girder. The wheel loads will be modeled as a moving load case.

MOVING LOAD CASE	DESCRIPTION	HORIZONTAL (Kips/girder)	VERTICAL (Kips/wheel)	MOMENT (k-ft/girder)
2 max axial	(2) HL93 max axial MPF = 1.0	-0.92 x 2 / 6 = -0.3	99.09 / 2 = 49.5	-25.34 x 2 / 6 = -8.4
4 max axial	(2) P15 max axial MPF = 1.0	-5.41 x 2 / 6 = -1.8	290.67 / 2 = 145.3	-148.92 x 2 / 6 = -49.6

2.3 Centrifugal Forces (LRFD 3.6.3)

- Centrifugal force is a function of the highway design speed and the radius of curvature of traffic lanes. The force is applied 6' above the roadway surface in the outer radial direction.

Radial force = C x (axle weights of the design truck or tandum)

$C = f v^2 / (gR) = 1.33 \times (95)^2 / (32.2 \times 475.7) = 0.79$

f = 4/3 for load combo other than fatigue and 1.0 for fatigue
v = highway design speed (ft/sec) = 65 mph = 95 ft/sec
g = gravity (32.2 ft/sec^2)
R = radius of curvature of traffic lane (ft) = 475.7 ft

Consider 2 lanes, MPF = 1.0
Radial force = 0.79 x 2 (32 + 32 + 8) = 114 kips

Since the bridge curvature ends between bents 2 and 3, there will be no centrifugal forces considered at bent 5.

2.4 Braking Force (LRFD 3.6.4)

This force is the greater of:

- 25% of the design truck or design tandem or,
- 5% of the truck/tandem plus lane load.

Apply the load 6' above the roadway.
Two lanes will be considered for braking load. MPF = 1.0

25% of truck/tandem = 0.25 x 2 (32 + 32 + 8) = 36 kips
5% of truck/tandem plus lane load = 0.05 x (144 + 2 x 0.64 x 413.4) = 34 kips

- This load should be distributed to the substructure based on a global model that accounts for the relative stiffness of the members and moment distribution. For simplicity, assume that the bents are all fixed at the bridge soffit. The load distribution to bent 5 will then be:

  Distribution factor = $(27.5)^{-3} / [(27.6)^{-3} + (31.5)^{-3} + (30.7)^{-3} + (27.5)^{-3}] = 0.29$
  0.3 x 36 = 10.8 kips => 1.8 kips/girder
  M = 10.8 x 27.5 = 297 k-ft => 49.5 k-ft/girder

2.5 Wind Load (LRFD 3.8)

- Tables are available for wind pressure based on base wind velocity of 100 mph.

Table 3.8.1.2.1-1 Base Pressures, P_B Corresponding to V_B = 100 mph.

SUPERSTRUCTURE COMPONTENT	WINDWARD LOAD, ksf	LEEWARD LOAD, ksf
Trusses, Columns, and Arches	0.050	0.025
Beams	0.050	NA
Large Flat Surfaces	0.040	NA

Table 3.8.1.2.2-1 Base Wind Pressures, P_B, for Various Angles of Attack and V_B = 100 mph.

Skew Angle of Wind	Trusses, Columns and Arches		Girders	
	Lateral Load	Longitudinal Load	Lateral Load	Longitudinal Load
Degrees	ksf	ksf	ksf	ksf
0	0.075	0.000	0.050	0.000
15	0.070	0.012	0.044	0.006
30	0.065	0.028	0.041	0.012
45	0.047	0.041	0.033	0.016
60	0.024	0.050	0.017	0.019

- Design wind velocity requires adjustment per equation 3.8.1.1-1 for wind on components >30 feet above ground.

$$V_{DZ} = 2.5V_0\left(\frac{V_{30}}{V_B}\right)\ln\left(\frac{Z}{Z_0}\right)$$

- Wind velocity, V_{30}, may be adjusted based on local conditions (ASCE 7 or site-specific surveys).

- May assume that wind velocity, V_{30}, is equal to base wind velocity of 100 mph.

- This bridge is approximately 30' high and local wind velocities are known to be less than 100 mph. Therefore, wind load for this bridge will use the default wind pressures.

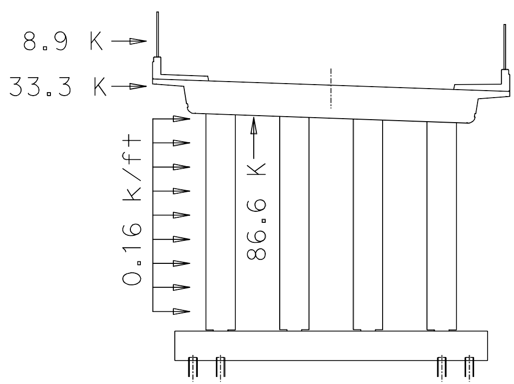
- Wind direction for design shall be that which produces the extreme force effect on the component. Assume that wind acting normal to the bridge will produce the extreme force effect.

Wind on superstructure = **50 psf** x 7.48 = 374 lb/ft > 300 lb/ft minimum
Wind on substructure = **40 psf**
Wind on vehicles = **100 lb/ft** applied 6' above deck
Vertical wind pressure = **20 psf** x deck area applied upward at quarter deck width (Strength III and Service IV combinations only)

Transverse wind load on structure = 0.374 x (99.74 + 78.08) / 2 = 33.3 kips applied 3.74' above soffit
And .04 x 4 = 0.16 kips/ft applied uniformly to column

Transverse wind load on vehicles = 0.100 x (99.74 + 78.08) / 2 = 8.9 kips applied 6' above deck

Vertical wind load = 0.02 x 48.72 x (99.74 + 78.08) / 2 = 86.6 kips applied 12.2' from edge of deck



Wind load on bent

2.6 Temperature (LRFD 3.12 & Caltrans Amendment 3.4.1 & 3.12)

- AASHTO allows temperature extremes to be determined using a table shown below (Procedure A) or maps (Procedure B); however, Caltrans only allows use of Procedure A.

Table 3.12.2.1.1-1 Procedure A Temperature Ranges.

CLIMATE	STEEL OR ALUMINUM	CONCRETE	WOOD
Moderate	0° to 120°F	10° to 80°F	10° to 75°F
Cold	−30° to 120°F	0° to 80°F	0° to 75°F

Concrete in moderate climate has Tmin = 10° F and Tmax = 80° F

- Temperature range for force effects formula given in Caltrans amendments

  $\Delta = \alpha L(T - T_{BaseConstr})$

- There is no guidance given by AASHTO or Caltrans in assuming the base construction temperature. It makes sense that the base construction temperature is close to the air temperature that the structure is built at, however, this could vary from 50° F to 100° F in the San Bernardino area. A good mean temperature would be more like 70° F. This would give a temperature fall of 60° F. The table below from Caltrans Bridge Design Specifications, April 2000, section 3.16 specified a temperature rise or fall ΔT = 35° F for moderate climate in California. If we used AASHTO with a realistic base construction temperature of 70° F, this would increase the temperature loading by 60/35 = 71%, which is not reasonable based on current practice.

Air Temperature Range	Design Range	
	Steel	Concrete
Extreme: 120° F Certain mountain and desert locations	Rise or Fall 60° F Movement/Unit Length .00039	Rise or Fall 40° F Movement/Unit Length .00024
Moderate: 100° F Interior Valleys and most mountain locations	Rise or Fall 50° F Movement/Unit Length .00033	Rise or Fall 35° F Movement/Unit Length .00021
Mild: 80° F Coastal Areas, Los Angeles, and San Francisco Bay Area	Rise or Fall 40° F Movement/Unit Length .00026	Rise or Fall 30° F Movement/Unit Length .00018

Use ΔT = 35° F consistent with Caltrans Bridge Design Specifications, April 2000, section 3.16. This will be confirmed with the Caltrans technical experts.

- Caltrans requires the forces due to temperature effects to be calculated using gross section properties and the lower bound load factor of 0.5 to account for cracking of the concrete columns (Caltrans Amendment 3.4.1).

- The longitudinal load at the bent due to temperature change should be determined using a global model to account for moment distribution effects with the superstructure. CONBOX was used to determine the forces shown below for a temperature fall of 35 degrees.

Case	Longitudinal Force (kips)	Vertical Force (kips)	Bending Moment (k-ft)
ΔT = -35° F	75.72	-3.71	2084.34
Load per girder	12.6	-0.6	347.4

- The in-plane load at the bent due to temperature can be determined using a local bent model with ΔT = 35° F. This should not be significant unless the bent is wide, however, it will be included since we already are neglecting the in-plane loads due to creep and shrinkage.

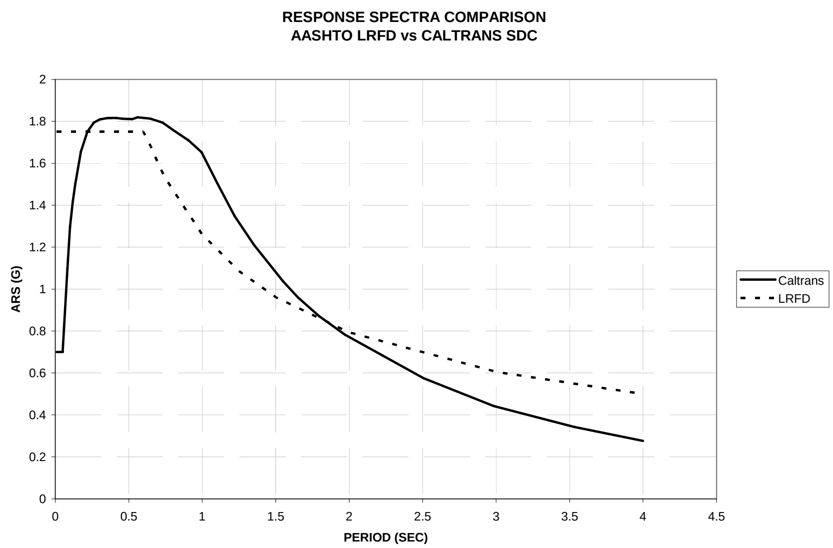
2.7 Earthquake (LRFD 3.10 and Caltrans SDC)

- AASHTO LRFD requires earthquake loads to be determined using modal analysis methods with response spectra and response modification factor. For a CIP/PS box girder structure in high seismic zone, the structure is designed to withstand the maximum forces developed from the inelastic behavior of the columns.

- Caltrans uses their Seismic Design Criteria (SDC) for ordinary standard bridges rather than the LRFD approach. SDC requires earthquake demand displacement to be determined using modal analysis with response spectra. The structure displacement demand must be less than the inelastic displacement capacity and the target displacement ductility should be satisfied. For a CIP/PS box girder structure, the structure is designed to withstand the maximum forces developed from the inelastic behavior of the columns.

- A comparison of the LRFD and Caltrans procedures is shown below.

ITEM	LRFD	CALTRANS
Key design assumption	Response modification factor used to size columns	Equal displacement observation theory and target displacement ductility used to size columns
Elastic demand analysis	(LRFD 4.7.4) Global response spectra analysis to determine forces	(SDC 2.2) Global response spectra analysis to determine displacements
Response modification factor	(LRFD 3.10.7.1) R = 3.5 for essential bridge and R = 5.0 for other bridge.	(SDC 2.2.4 & 3.1.4.1) SDC uses target and local displacement ductility rather than response modification factor. Target ductility $\mu_D = 5$ Local ductility $\mu_C = 3$
Response spectra	(LRFD 3.10.6.1) $C_m = \frac{1.2AS}{T_m^{2/3}} \le 2.5A$ See curve below	(SDC 2.1.1) Response spectra curves from ATC-32, modified as needed for fault characteristics. See curve below
Inelastic demand analysis	(LRFD 3.10.9.4) Pushover analysis using $1.3M_n$ for columns	(SDC 4.3.1) Pushover analysis using overstrength moment for columns
Capacity of members	Use concrete capacity as with any other loading	(SDC 3) Use increased concrete capacity for seismic loading
Design approach	• Perform global response spectra analysis and size column by reducing demand forces by response modification factor. • Perform pushover analysis to determine loads on foundation and superstructure.	• Perform global response spectra analysis and size column by reducing demand forces using target displacement ductility. • Perform pushover analysis to determine loads on foundation and superstructure. • Verify that pushover displacement is less than demand displacement • Verify target displacement ductility and local displacement ductility is satisfied



Period (sec)	ARS (G's)
0.01	0.70
0.05	0.70
0.08	1.10
0.10	1.30
0.12	1.42
0.14	1.50
0.17	1.66
0.22	1.75
0.26	1.79
0.30	1.81
0.35	1.82
0.38	1.82
0.42	1.82
0.46	1.81
0.52	1.81
0.56	1.82
0.65	1.81
0.73	1.79
0.80	1.76
0.91	1.71
1.00	1.65
1.10	1.50
1.22	1.35
1.35	1.21
1.55	1.04
1.65	0.96
1.79	0.87
1.97	0.78
2.51	0.58
2.98	0.44
3.53	0.34
4.00	0.28

Use Caltrans curve for magnitude 7.25 event with peak ground acceleration of 0.7G and soil profile D. Curve is modified for near fault effects.

- The Caltrans procedure for seismic design is more complicated, since it requires displacement calculations and the use of different material properties for capacity and inelastic demand. Both procedures have limitations, as the Caltrans SDC is based on the “equal displacement observation” and target displacement ductilities while the LRFD is based on using response modification factors.

2.8 Load combinations (LRFD 3.4 and Caltrans Amendment 3.4.1)

See Caltrans amendments for load combinations. The following combinations will be used.

Combo	DC	DW	PS	HL93	P15	WS	WL	TU
Strength IA	1.25	1.5	0.5	1.75	0	0	0	0.5
Strength IIA	1.25	1.5	0.5	0	1.35	0	0	0.5
Strength IIIB	0.9	1.5	0.5	0	0	1.4	0	0.5
Strength VA	1.25	1.5	0.5	1.35	0	0.4	1.0	0.5
Service I	1.0	1.0	0.5	1.0	0	0.3	1.0	0.5
Service II	1.0	1.0	0.5	1.3	0	0	0	0.5

Notes:

1. Use uplift wind load for strength IIIB
2. Service load conditions use 0.5 factors for PS and TU rather than 1.0 since these combinations are for investigation of service load stresses in the bent cap.

SECTION 3.0 GLOBAL ANALYSIS

3.1 MODEL SELECTION

- A global model may be required to determine some of the forces in the bent structure. The need for a global model depends on the complexity of the bridge, types of loadings and method used to analyze the superstructure.
- The CONBOX program was used to analyze the superstructure for this example. The loads at the bent due to superstructure dead load, live load, secondary prestress forces, creep and shrinkage forces and temperature forces were determined using CONBOX.
- The loads at the bent due to centrifugal forces, braking and wind load were determined from manual calculations for this example.
- The global model is therefore only needed to determine the demand at the bent due to earthquake.
- Several programs are available to perform global analysis as shown below.

Program	Description
SAP2000	General finite element program
SEISAB	Program designed specifically for seismic analysis of bridges. This program has not been updated for Caltrans SDC requirements, but still may be used to determine the global displacement demand and perform preliminary sizing of the columns.
GT-Strudl	General finite element program
STAAD	General finite element program

SAP2000 will be used for the global model.

3.2 MODELING GUIDELINES

- Modeling guidelines for elastic dynamic analysis (SDC 5.2.2)

  1. Minimum 3 elements per column and 4 elements per span
  2. Use complete quadratic combination method (CQC)
  3. Need 90% mass participation

- Section properties (SDC 5.6.1)

  1. Columns use Ieff = Icr (XSECTION or CONSEC)
  2. Superstructure use Ieff = 0.50 Ig to 0.75 Ig based on amount of reinforcement

  Note: SDC 5.7 recommends using reduced Ieff for temperature and shrinkage loads, however, Caltrans Amendment 3.4.1 to AASHTO LRFD requires use of Ig and load factor of 0.5.

- Abutment stiffness (SDC 7.8)

  Longitudinal:

  Stiffness $K_{abut} = K_i \times w \times h / 5.5$ (k/in)

  K_i = 20 k/in/ft
  w = backwall/diaphragm width (ft)
  h = backwall/diaphragm height (ft)

  Capacity P = w x h x 5 x h / 5.5

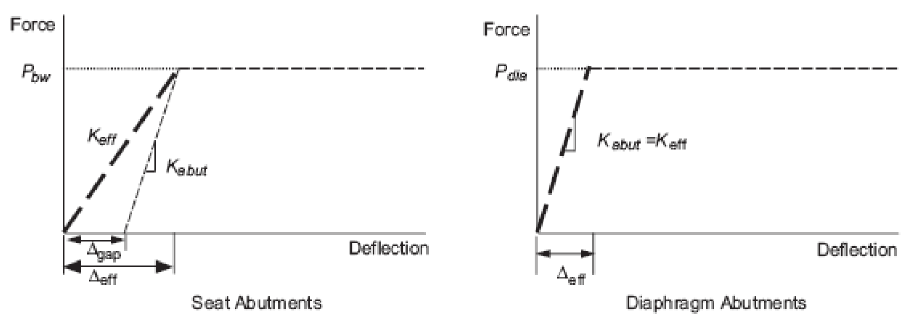


Figure 7.14A Effective Abutment Stiffness

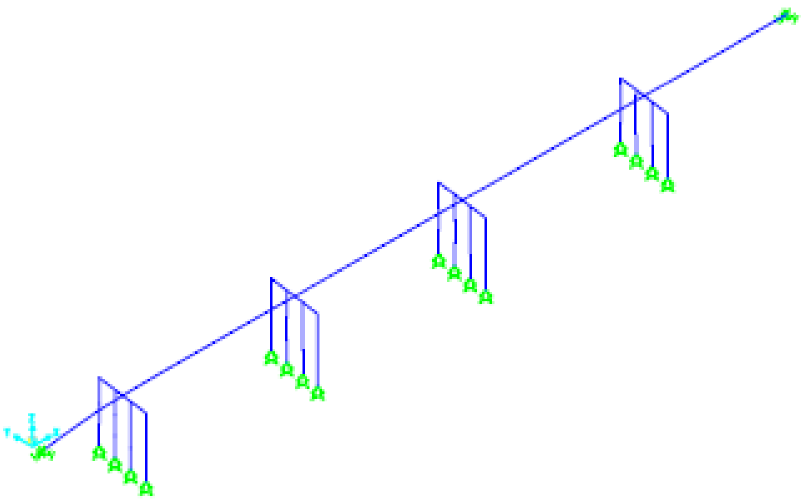
K_{eff} shall include the effect of the abutment gap (joint) and be used in the initial analysis. If the analysis determines that the abutment capacity is exceeded by more than a factor of 2, then the spring needs to be reduced linearly down to a minimum spring of 0.1 x K_{eff} at an overcapacity factor of 4.

Transverse:

Transverse stiffness may be calculated using a realistic force-deflection relationship for the abutment system. Otherwise use a transverse abutment stiffness equal to 50% of the adjacent bent stiffness.

3.3 MODEL

- The SAP2000 model for this bridge is shown below. The bent cap should be modeled with a large lateral bending stiffness to prevent bending deformation in the plane of the deck.



3.4 ANALYSIS RESULTS

- For a detailed description of using SAP2000 to perform response spectra analysis, see "Earthquake Analysis with SAP2000" course notes available on the Technical Software intranet site for SAP2000 at http://dmjmharrisportal.aecomnet.com/sites/tech_soft/structural/sap2000/default.aspx

- The maximum global demand displacements are shown below.

Longitudinal displacement = 23.97"
Transverse displacement = 20.66"

SECTION 4.0 LOCAL ANALYSIS

4.1 MODEL SELECTION

- A local model is required to determine the forces in the bent.
- Validated software is available to perform local analysis as shown below. This software has been verified to perform as intended. Users should complete the following steps when using software.
  - Read the manual or receive training to understand the software use and limitations
  - Verify software input after it is entered, preferably with graphical representation
  - Verify software output (equilibrium of forces and reactions, reasonableness of results like deflected shape)

Program	Description
SAP2000	General finite element program
RCPIER	Program designed specifically for bent analysis and design. This program does not include the Caltrans amendments to LRFD or the Caltrans SDC requirements, but still may be used to determine the effects of most of the bent loadings.
GT-Strudl	General finite element program
STAAD	General finite element program
XSECTION	Column moment curvature
CONSEC	Column moment curvature

SAP2000 will be used for the local model.

4.2 MODELING GUIDELINES

- Nodes and elements

  1. Provide bent cap nodes at center of spans to facilitate output moment at this location
  2. Provide bent cap nodes at center and faces of columns to model stiffer joint area
  3. Minimum 3 elements per column to be consistent with global model
  4. Provide column nodes at center and top of footing to model stiffer joint area
  5. Provide column nodes at center and soffit of bent cap to model stiffer joint area

- Section properties (SDC 5.6.1)

  1. Columns use Ieff = Ig, except use Icr for pushover analysis
  2. Superstructure use Ieff = Ig, except use 0.50 Ig to 0.75 Ig based on amount of reinforcement for pushover analysis
  3. Use factor of 10 on section properties to model stiffer joint area

  Note: SDC 5.7 recommends using reduced Ieff for temperature and shrinkage loads, however, Caltrans Amendment 3.4.1 to AASHTO LRFD requires use of Ig and load factor of 0.5.

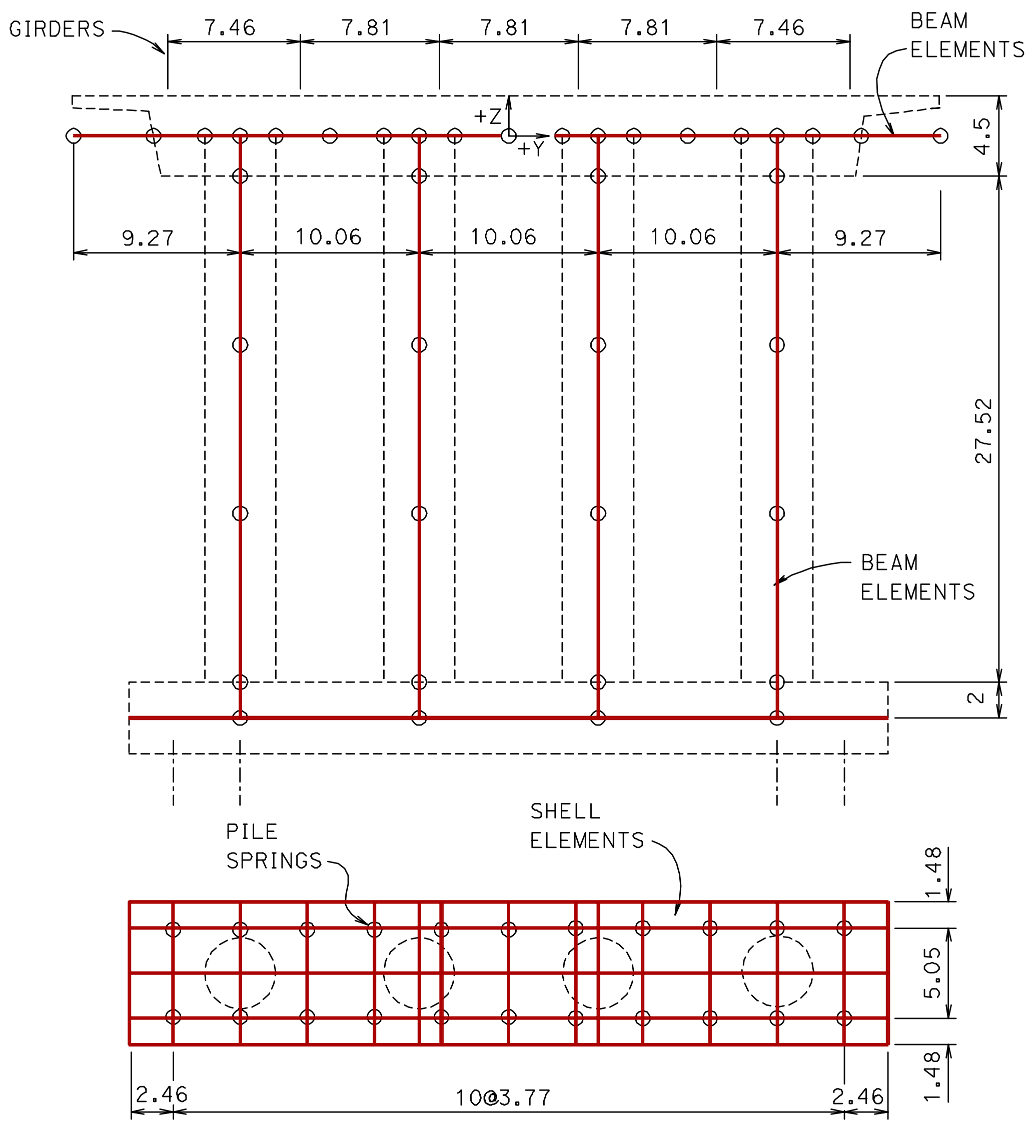
SHEET 4-2 OF

JOB TITLE **BENT DESIGN** ORIGINATOR Bob Matthews DATE 10/19/2007

JOB No. CALCULATION No. REVIEWER DATE

4.3 MODEL

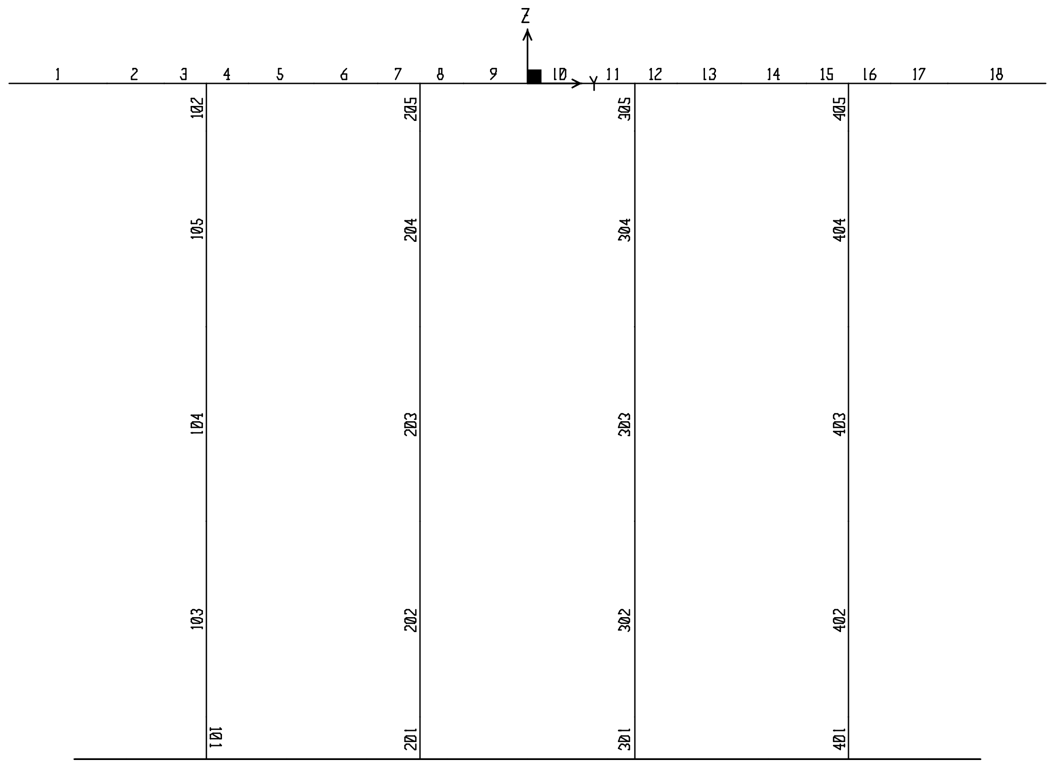
- Geometry:



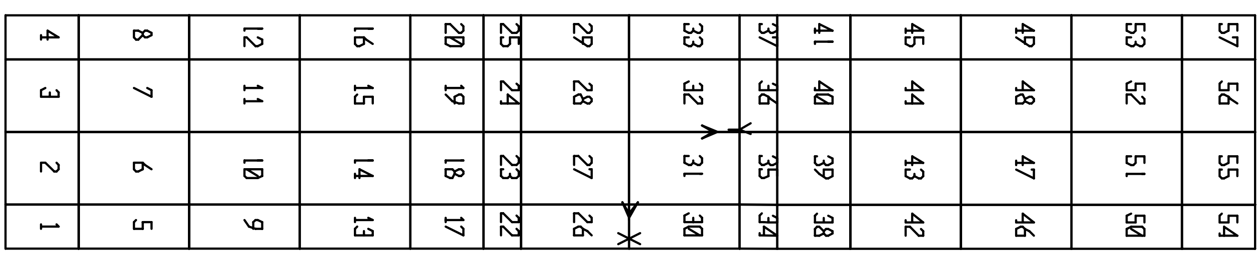
SHEET 4-3 OF

JOB TITLE **BENT DESIGN** ORIGINATOR Bob Matthews DATE 10/19/2007

JOB No. CALCULATION No. REVIEWER DATE



FRAME ELEMENTS



FOOTING ELEMENTS

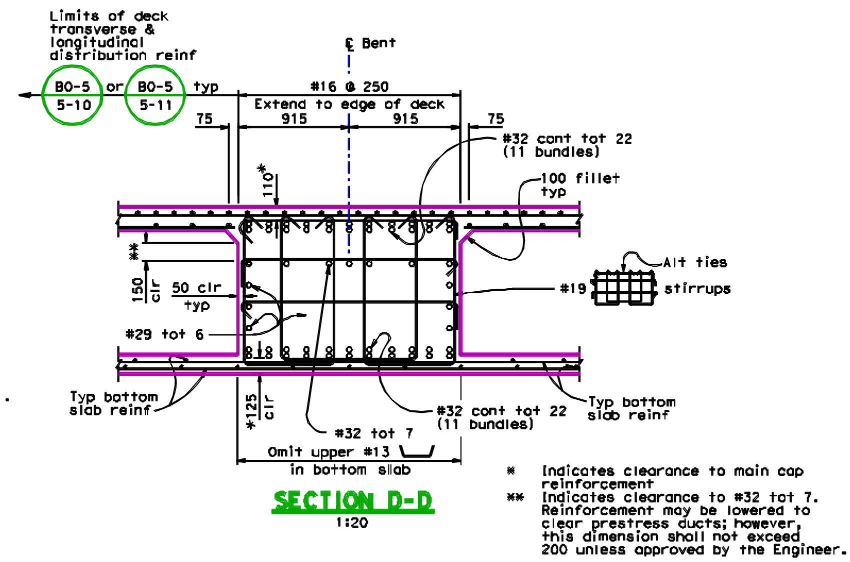
- Restraints

The following restraints will be applied to the model.

1. 3D springs at piles
2. Bent cap restrained for bending about Y axis (to keep model stable and allow longitudinal loads input)

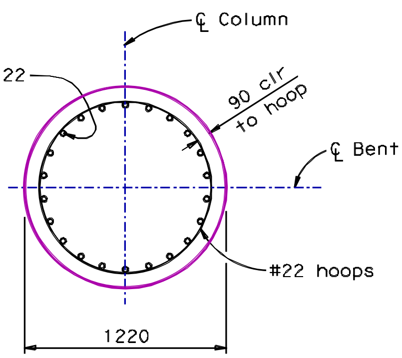
- Member Properties:

    1. Model bent cap as rectangular section 6' wide x 4.5' deep



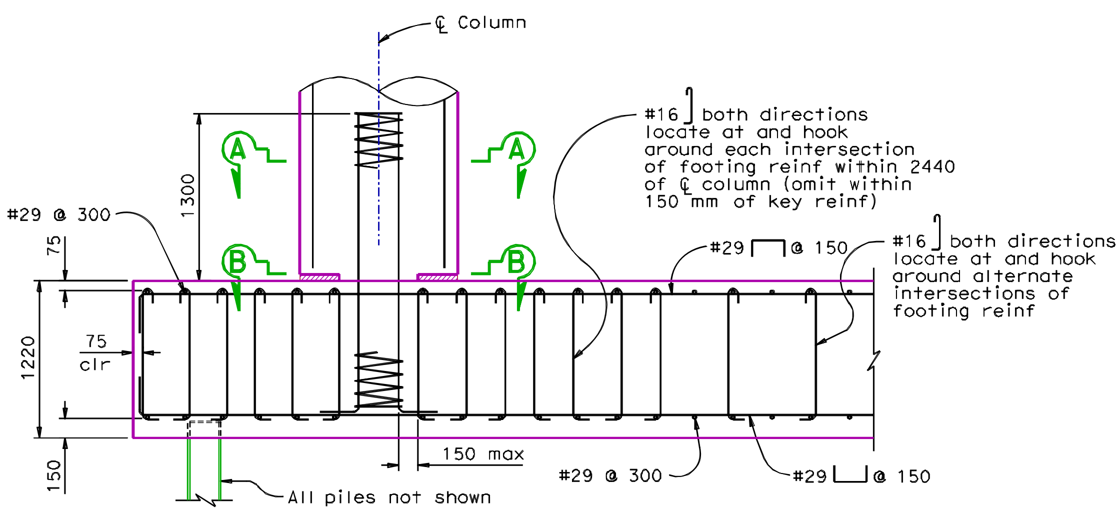
For joint area use Ix (effective) = 10 Ix (gross)
For pushover model use Ix (effective) = 0.75 Ix (gross)

2. Model column as circular section 4 ft diameter



For joint area use Ix (effective) = 10 Ix (gross)
For pushover model use section designer section with Ix (effective) = Ix (crack)
Ix (crack) = 0.25 Ix (gross) (from preliminary m-ϕ analysis)
And J (effective) = 0.2 J (gross)

3. Model footing as shell 4' thick



FOOTING DETAIL

Use Ix (effective) = 10 Ix (gross) so footing will act more rigid

4. Model piles with springs

Vertical capacity is 280 kips (70 ton pile) with 1 inch deflection => Kz = 280 k/in
Use Kz = 2800 k/in in pushover model to reduce foundation deflection

Lateral Y capacity ~ 20 k/in + 8 x 4 x 5 / 22 => Ky = 27 k/in (includes passive on footing)
Use Ky = 270 k/in in pushover model to reduce foundation deflection

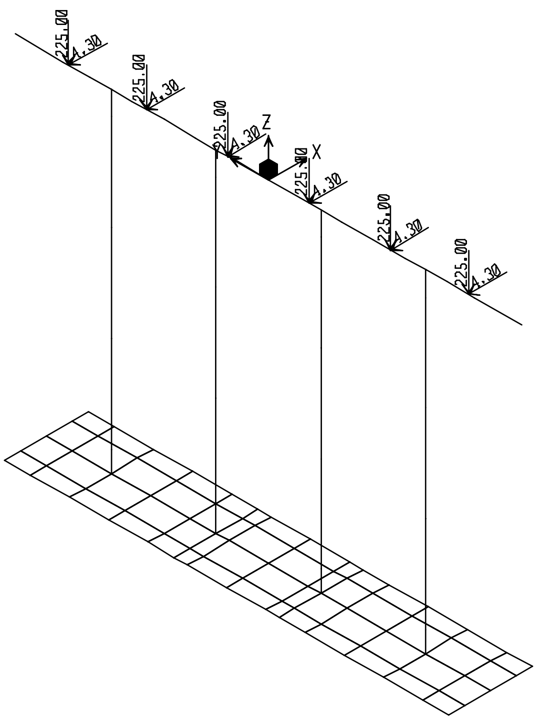
Lateral X capacity ~ 20 + 42.6 x 4 x 5 / 22 => Kx = 59 k/in (includes passive on footing)
Use Kx = 590 k/in in pushover model to reduce foundation deflection

Notes:

a. Foundation deflection was not included in demand model and should be limited in the capacity model.
b. Vertical pile stiffness estimated based on 1” deflection at ultimate capacity
c. Lateral pile stiffness taken from Caltrans 2000 BDS section 4
d. Passive pressure on pile cap assumes 5 ksf and 1” deflection.

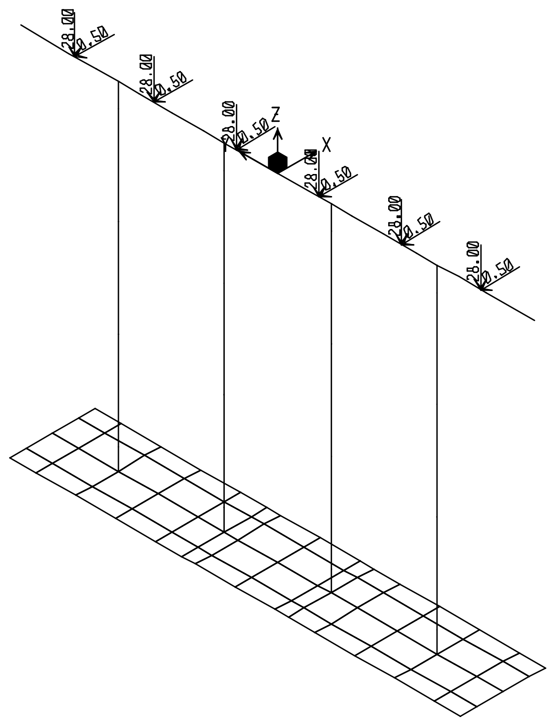
- Loads:

The loads computed in Section 2.0 are applied to the model as shown below.



DC - Dead load (constant)

Load includes 240 psf on footing



DW – Dead load (varying)

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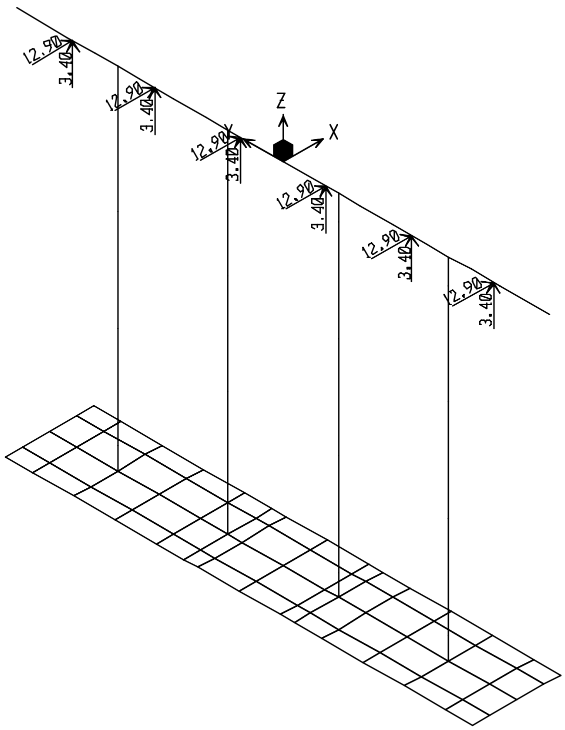
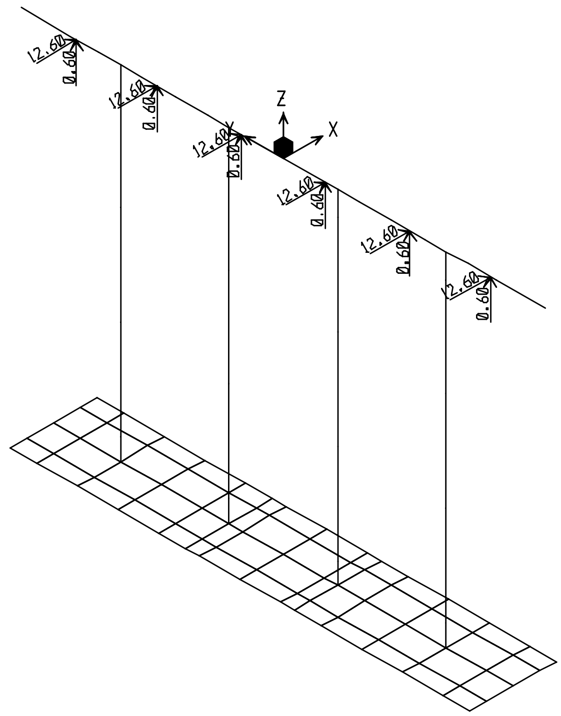


PS – Prestressing



TU – Temperature

A uniform ΔT = 35 degrees is also applied directly to the bent model

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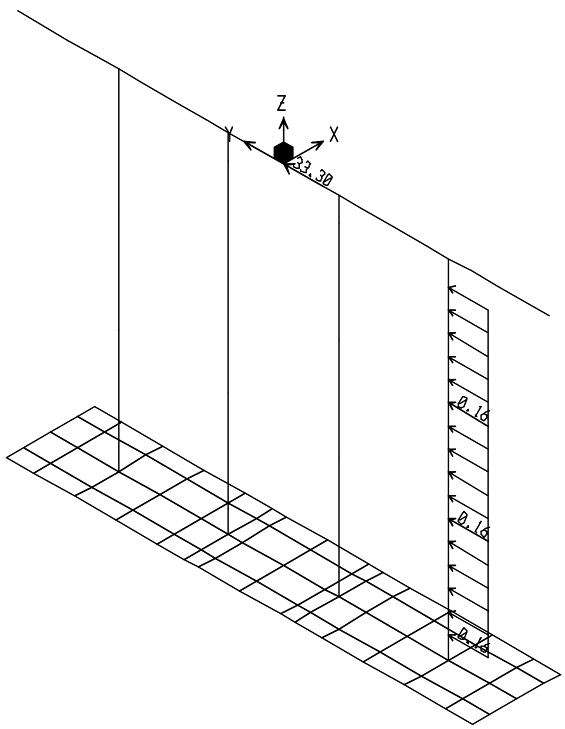
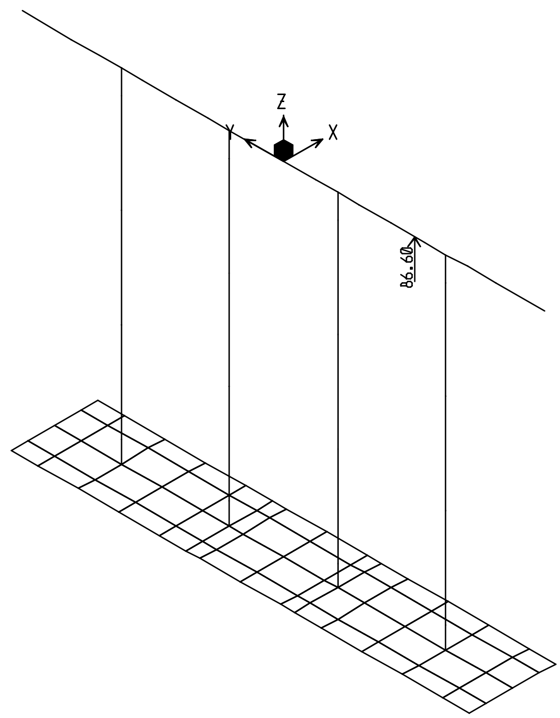
JOB TITLE **BENT DESIGN**		ORIGINATOR Bob Matthews	DATE 10/19/2007
JOB No.	CALCULATION No.	REVIEWER	DATE



WS – Wind on structure



WS-up – Wind on structure (up)

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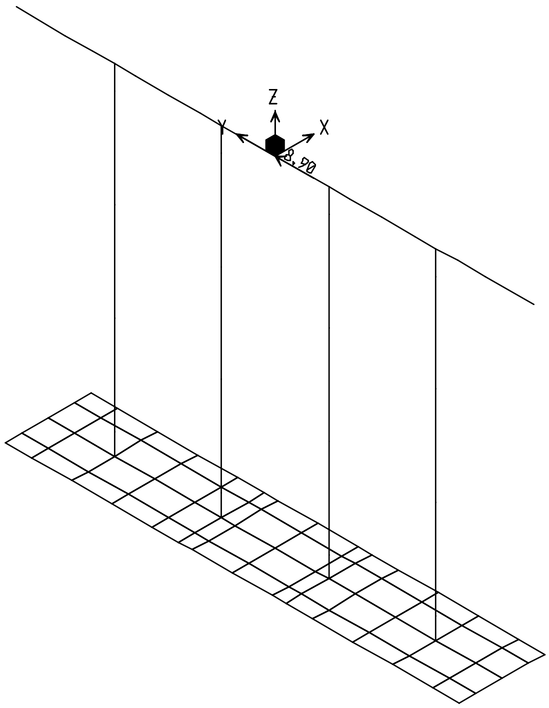
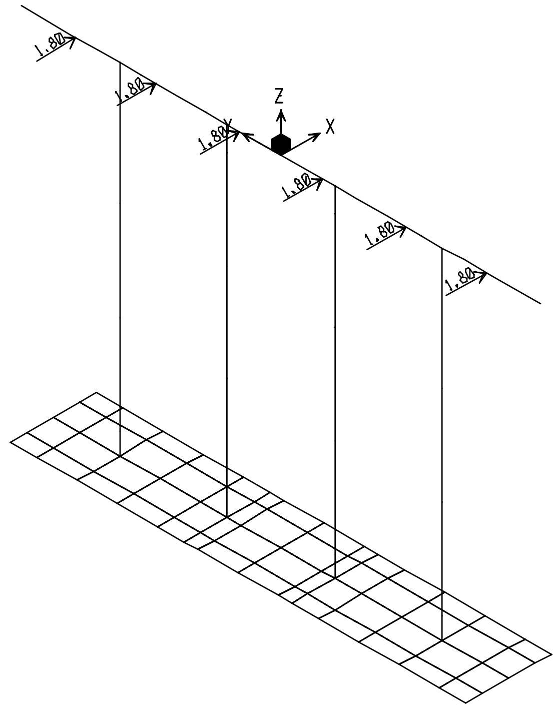
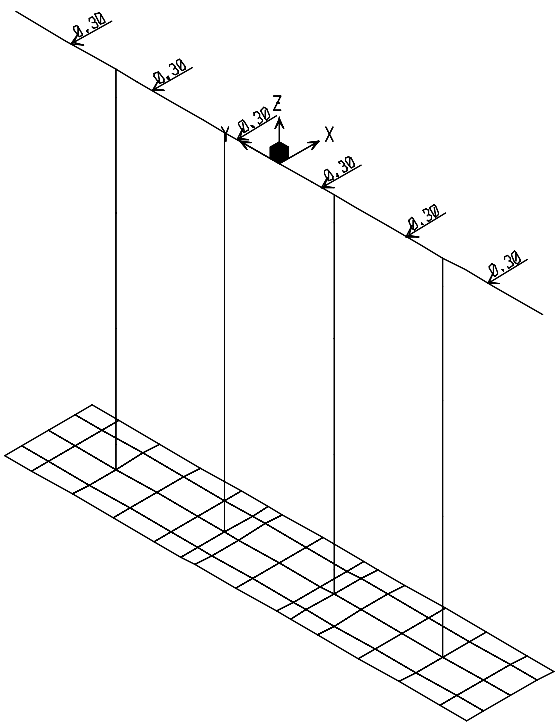
JOB TITLE **BENT DESIGN**		**ORIGINATOR** Bob Matthews	**DATE** 10/19/2007
JOB No.	**CALCULATION No.**	**REVIEWER**	**DATE**



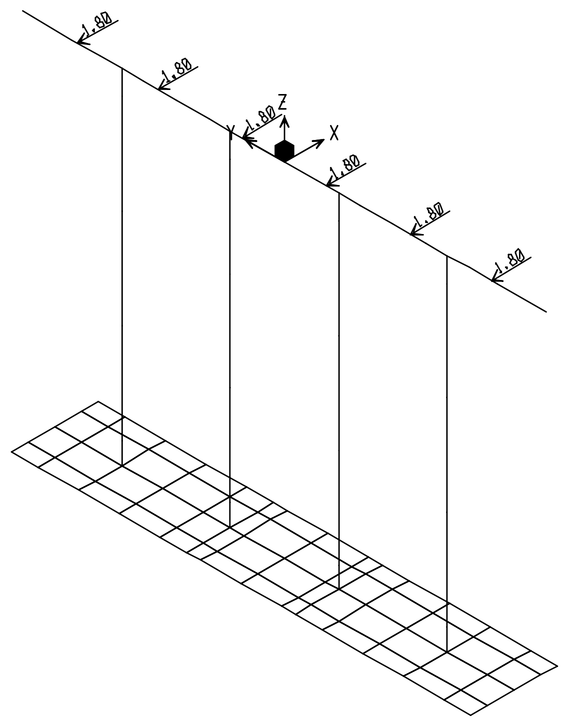
WL - Wind on live load



BR – Braking



HL93 – longitudinal



P15 – Longitudinal

The vertical loads are applied as moving loads

4.4 ANALYSIS RESULTS

Model has been checked for input, equilibrium of forces and reasonableness of output.

- Bent cap

The critical loads on the bent cap are shown below.

CASE	ELEMENT	SHEAR (kips)	MOMENT-3 (k-ft)	COMMENT
STR-IIA	2	732	-2778	
	5	-559	-2907	
	10	163	485	
SER-II	5	308	1271	Max service load

- Column

The critical loads on the column are shown below.

CASE	ELEMENT	AXIAL (kips)	SHR-2 (kips)	MOM-3 (k-ft)	SHR-3 (kips)	MOM-2 (k-ft)
STR-IIA	105	1379	17.5	481	6.3	174
DC	105	467	3.5	96.1	6.5	178

- Footing

The area elements show high loads concentrated at the outer columns. The code allows the forces to be determined through sections at the face of pins. You can either determine these forces using the pile reactions and manual calculations or use SAP2000 section cut feature to sum the forces. The critical loads on the footing from SAP2000 section cuts are shown below.

CASE	SECTION	V (kips)	M (k-ft)	V (kips/ft)	M (k-ft/ft)
STR-IIA	LONG1	693	914	58.2	76.8
STR-IIA	SHORT1	447	1720	55.9	191
STR-IIA	SHORT2	572	601	71.5	75.1

- Piles

The critical loads on the piles are shown below.

CASE	JOINT	HORIZ-X (kips)	HORIZ-Y (kips)	AXIAL (kips)	COMMENT
SER-I	138	-0.3	-6.7	133	LRFD 10.5.2.2 settlement
SER-II	68	-3.7	5.8	142	LRFD 10.5.2.2 lateral deflection
STR-IIA	68	-2.8	5.8	251	
STR-IIIB	136	-3.9	-8.2	110	

4.5 NONLINEAR STATIC (PUSHOVER) ANALYSIS

- AASHTO LRFD 3.10.9.4 requires pushover analysis for bents in high seismic regions using an overstrength moment = $1.3M_n$.

- For a detailed description of using SAP2000 to perform pushover analysis using $1.3M_n$, see "Earthquake Analysis with SAP2000" course notes available on the Technical Software intranet site for SAP2000 at http://dmjmharrisportal.aecomnet.com/sites/tech_soft/structural/sap2000/default.aspx

- The Caltrans overstrength moment is less than 10% higher than the LRFD overstrength moment using the maximum axial load

Axial load	Mo (Caltrans SDC)	Mo AASHTO LRFD
937 kips	1.2 x 3424 = 4109	1.3 x 2907 = 3779

- Pushover analysis will be performed per Caltrans SDC to determine the deflection capacity and demands on the bent cap and footing.

- Longitudinal analysis

Longitudinal analysis may be performed using the CONSEC program with a constant dead load of 467 kips on the column. The results of this analysis are shown below.

```
* * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                   *
*          P R O G R A M   C O N S E C              *
*                                                   *
*                INPUT DATA ECHO                    *
*                                                   *
* * * * * * * * * * * * * * * * * * * * * * * * * * *
(Version  1.4)                  10/15/07, 9:08 am

Input file  = consec.in
Output file = consec.out

S T R E S S - S T R A I N   I N F O R M A T IO N
=====================================================

CONCRETE MODEL:

     Mander concrete model

          Unconfined strain      = 0.002
          Unconfined stress      = 5200
          Modulus of elasticity = 4110328
          Ultimate strain        = 0.02201
          Ultimate stress        = 0
          Spalling strain        = 0.005
          Confined stress        = 8413.7
```

```
          Circular hoop confinement:

               Bar diameter      = 0.875
               Long spacing      = 3
               Concrete cover    = 4
               Yield strength    = 60000
               Ultimate strain   = 0.09
               Hoop diameter     = 40

     REINFORCING MODEL:

         Park reinforcing model
         (with complex strain hardening)

              Point     Strain    Stress

               1        -0.09     -95000
               2        -0.0125   -68000
               3        -0.0023   -68000
               4         0.0023    68000
               5         0.0125    68000
               6         0.09      95000

     C O N C R E T E   C O N F I G U R A T I O N
     =====================================================


     1 CIRCULAR SECTIONS:

         Ycenter   Radius    Add

          24        24        1

     R E I N F O R C I N G   C O N F I G U R A T I O N
     =====================================================


     1 REINFORCING ARCS:

         Ycenter   Radius    Abeg      Atot      Nbar      Abar

          24        19        0         343.6364  22        1

     L O A D   C O N D I T I O N S
     =====================================================
     (Units = K-ft)

     1 LOAD CONDITIONS:

         No        Axial     Moment    Shear     Torsion

          1        467.0     0.0       0.0       0.0
```

```
M E M B E R   P R O P E R T I E S
=====================================================

MEMBER PROPERTIES:

    Member length                      = 330

BOUNDARY CONDITIONS:

    Condition           End i     End j

    Shear restraint       1         1
    Moment restraint      1         0

* * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                   *
*           P R O G R A M   C O N S E C             *
*                                                   *
*                   OUTPUT DATA                     *
*                                                   *
* * * * * * * * * * * * * * * * * * * * * * * * * * *

S E C T I O N   P R O P E R T I E S
=====================================================

GROSS CONCRETE SECTION:

     Area    = 1.8096E+03
     Ybar    = 2.4000E+01
     Io      = 2.6058E+05 About concrete CG

REINFORCING STEEL:

     Area    = 2.2000E+01
     Ybar    = 2.4000E+01
     Io      = 3.9710E+03 About reinf CG

TRANSFORMED CONCRETE SECTION:

     Area    = 1.9628E+03
     Ybar    = 2.4000E+01
     Inertia = 2.8824E+05

M O M E N T   C U R V A T U R E
=====================================================
Moments about centroid of gross concrete section
(Units = K-ft)

Load Condition Number 1
Axial Load = 467.0
```

Strain	c	Axial	Curvature	Moment
0.00050	18.95	465.3	0.000026	999.9
0.00100	15.60	462.1	0.000064	1807.9
0.00150	14.40	459.5	0.000104	2431.1
Reinf tens yield				
0.00200	13.25	463.8	0.000151	2667.4
0.00250	12.45	460.7	0.000201	2772.0
0.00300	11.90	463.5	0.000252	2817.3
0.00350	11.55	457.4	0.000303	2840.0
0.00400	11.30	458.4	0.000354	2851.9
0.00450	11.15	461.4	0.000404	2851.4
Reinf comp yield				
0.00500	11.15	455.8	0.000448	2839.8
0.00550	11.25	455.7	0.000489	2830.3
Conc spalling				
0.00600	11.40	461.1	0.000526	2824.5
0.00650	11.50	454.9	0.000565	2812.8
0.00700	11.60	461.2	0.000603	2826.1
0.00750	11.65	464.3	0.000644	2843.7
0.00800	11.65	459.5	0.000687	2857.7
0.00850	11.65	455.6	0.000730	2874.9
0.00900	11.65	452.3	0.000773	2894.2
0.00950	11.70	463.7	0.000812	2925.3
0.01000	11.70	463.1	0.000855	2946.5
0.01050	11.70	461.6	0.000897	2965.2
0.01100	11.70	461.3	0.000940	2984.8
0.01150	11.70	460.3	0.000983	3002.4
0.01200	11.70	459.7	0.001026	3020.2
0.01250	11.70	457.9	0.001068	3036.9
0.01300	11.70	456.3	0.001111	3053.3
0.01350	11.75	464.0	0.001149	3074.6
0.01400	11.75	461.9	0.001191	3087.5
0.01450	11.75	460.1	0.001234	3099.4
0.01500	11.75	457.7	0.001277	3109.7
0.01550	11.75	455.7	0.001319	3120.0
0.01600	11.75	455.3	0.001362	3131.6
0.01650	11.75	454.2	0.001404	3141.7
0.01700	11.80	465.8	0.001441	3161.9
0.01750	11.80	463.6	0.001483	3169.5
0.01800	11.80	462.9	0.001525	3178.5
0.01850	11.80	462.0	0.001568	3186.6
0.01900	11.80	460.8	0.001610	3194.0
0.01950	11.80	459.0	0.001653	3200.1
0.02000	11.80	456.2	0.001695	3204.6
0.02050	11.80	454.4	0.001737	3209.9
0.02100	11.80	453.3	0.001780	3215.6
0.02150	11.80	452.0	0.001822	3220.9
0.02200	11.85	466.1	0.001857	3240.1

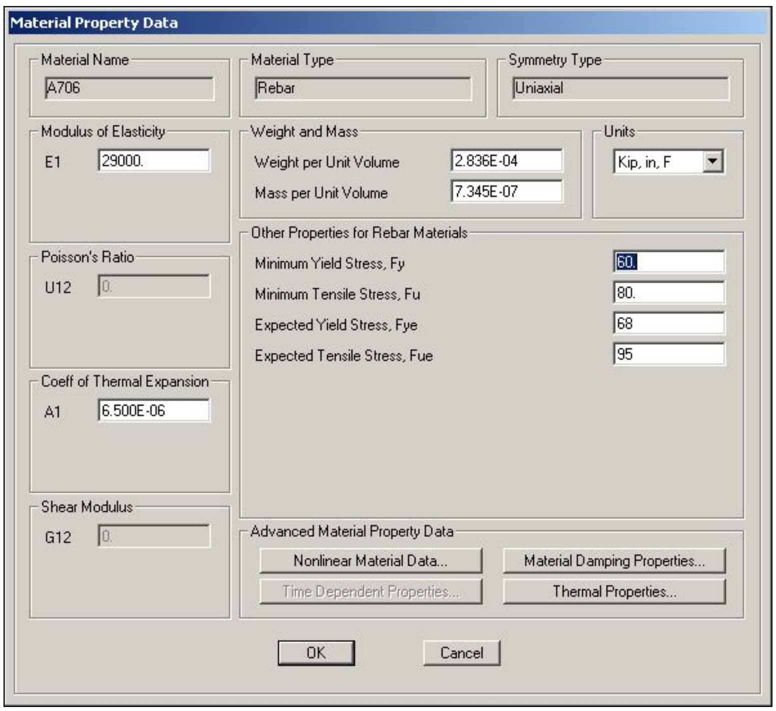
```
Yield curvature           = 1.0417E-04
Ultimate curvature        = 1.8565E-03
Idealized plastic moment  = 2991.3
Maximum tension           = -1335. At moment = 3240.1

Cracked moment of inertia = 6.8136E+04
Based on 1st yield strain = -0.00296

LOCAL MEMBER DUCTILITY:

    Idealized yield curvature = 1.2817E-04
    Plastic hinge length      = 37.9
    Yield deflection          = 4.653
    Ultimate deflection       = 25.033
    Local member ductility    = 5.4
```

- Transverse analysis

Transverse analysis may be performed using the SAP2000 program with dead load only. Effective section properties of the members will be used as described in section 4.3. The following procedure is used with SAP2000 to internally generate hinge properties based on Caltrans SDC requirements.

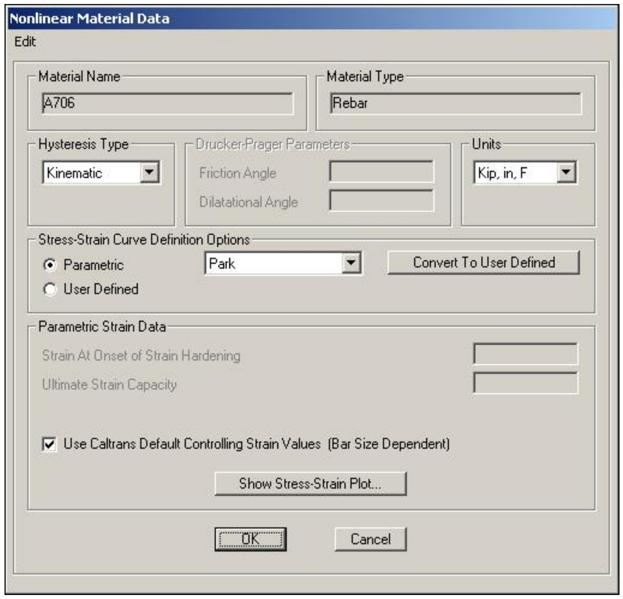
1. Define material properties for rebar

   – Define > Materials
   – Check “Show Advanced Properties”
   – Click “Add New Material Quick”
   – Select Rebar ASTM A706
   – Click “Modify/Show Material Properties”
   – Change expected yield stress and tensile stress per SDC 3.2.3 as shown below



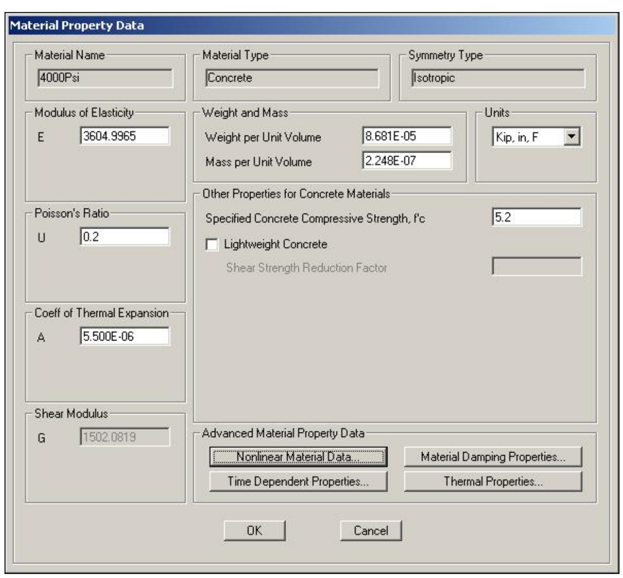
   – Select “Nonlinear Material Data”

– Select Park Stress-Strain Curve Definition Option and Check “Use Caltrans Default Controlling Strain Values” as shown below

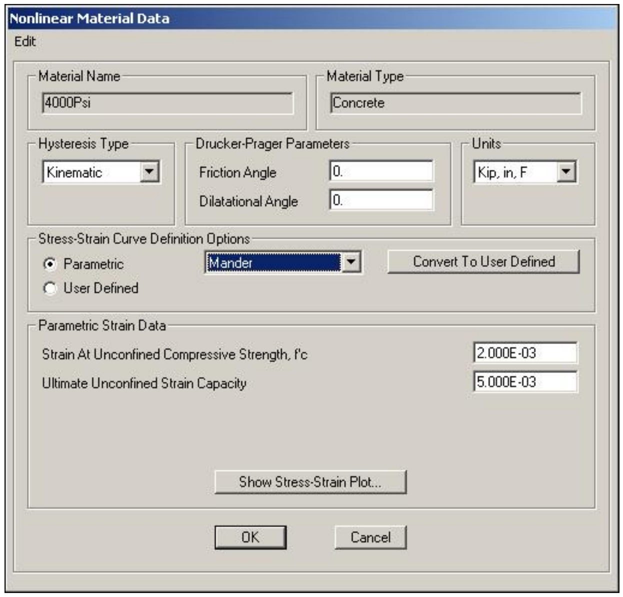


2. Define material properties for concrete

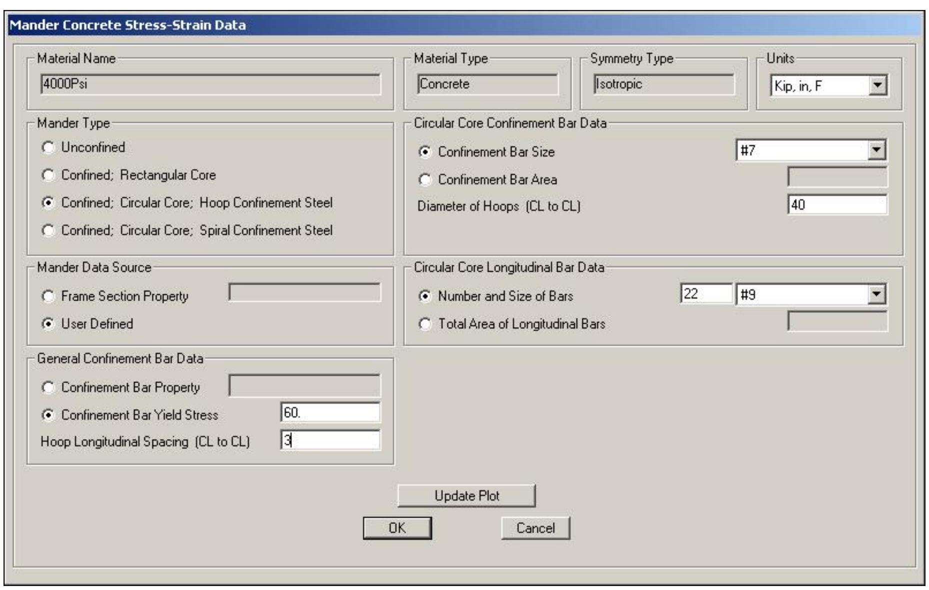
– Define > Materials
– Check “Show Advanced Properties”
– Click “Add New Material Quick”
– Select Concrete f’c 4000
– Click “Modify/Show Material Properties”
– Change specified concrete compressive strength per SDC 3.2.6 as shown below



- Select “Nonlinear Material Data”
- Select Mander Stress-Strain Curve Definition Option as shown below

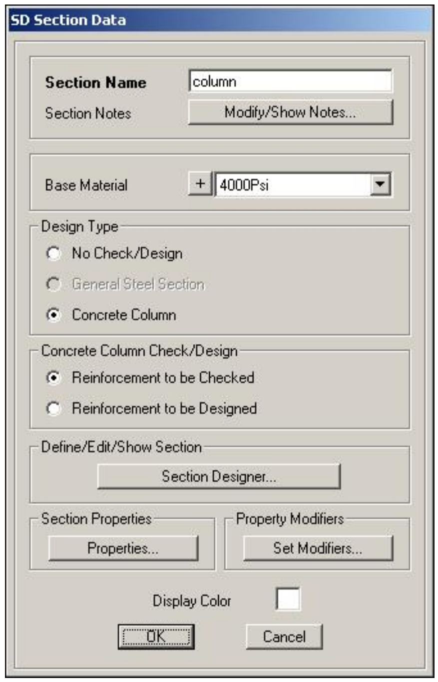


- Click “Show Stress-Strain Plot”
- Select “Modify/Show Mander Data” and complete data for configuration of column as shown below

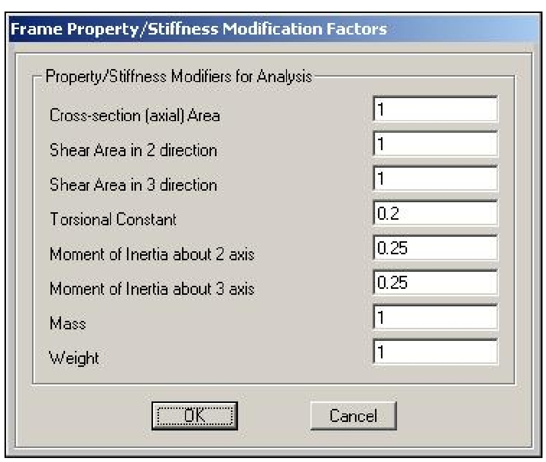


3. Define section designer section for column

- Define > Frame Section
- Click “Add New Property”
- Select “Other / Section Designer”
- Complete SD Data as shown below

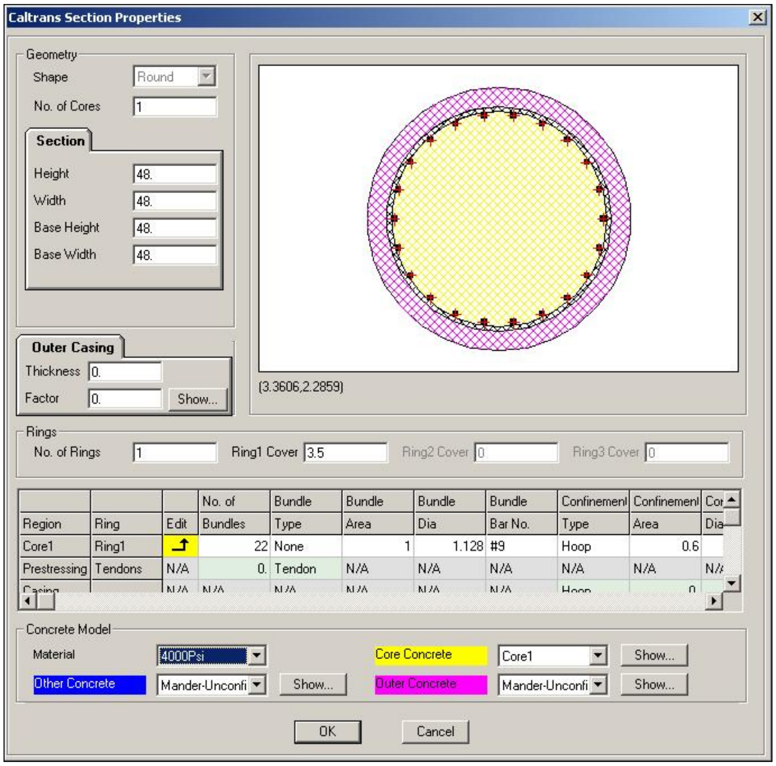


- Click “Property Modifiers” and complete data as shown below

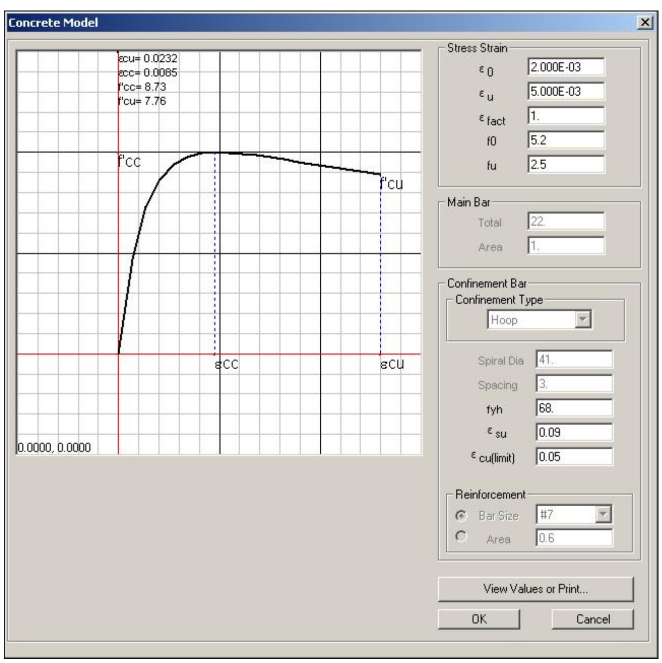


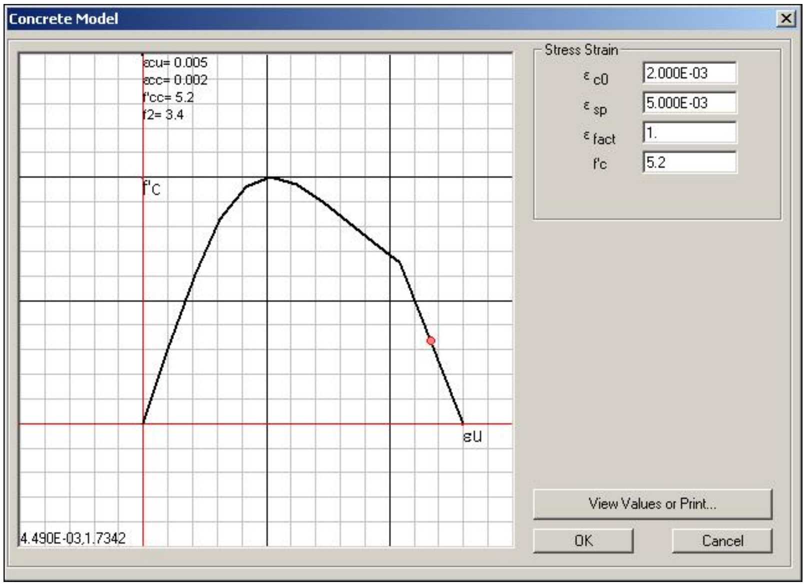
- Click “Section Designer” to start up section designer module

- Draw Caltrans Shape Round
- Right click on shape and complete data as shown below

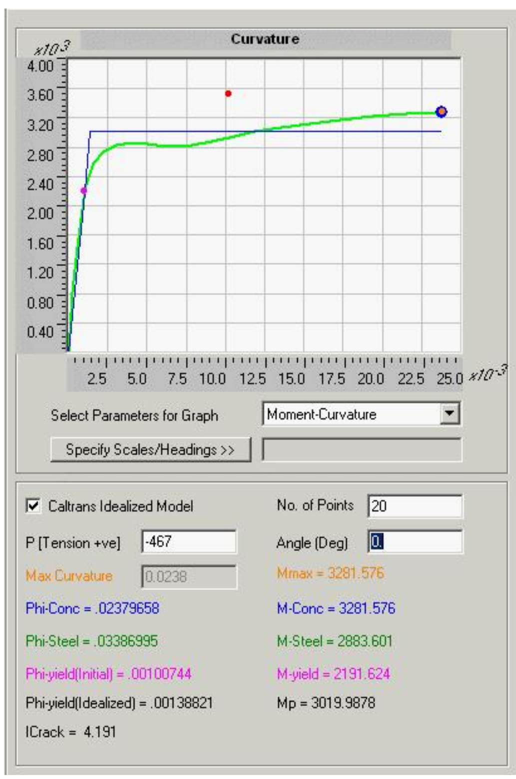


- Click "Show" for Core Concrete and Outer Concrete to verify data as shown below





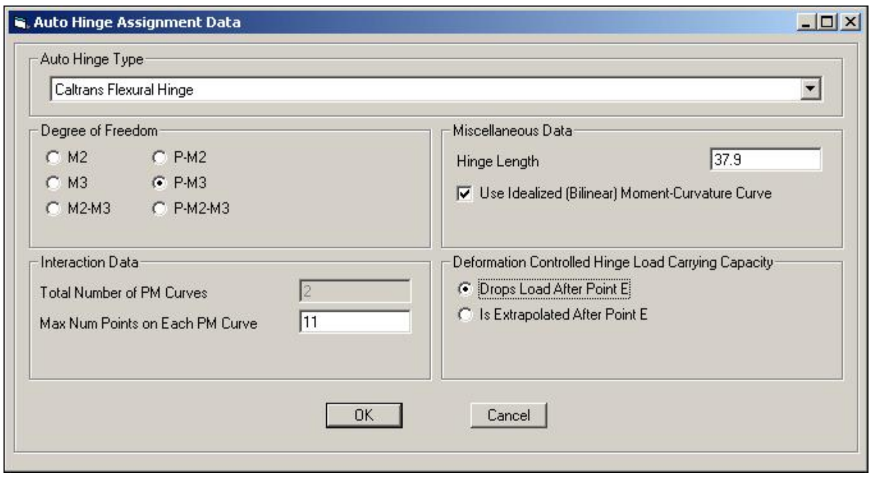
- Display > Show Moment-Curvature Data
- Check “Caltrans Idealized Model” and fill in axial load to check as shown below



4. Assign section to members

5. Assign frame hinges automatically

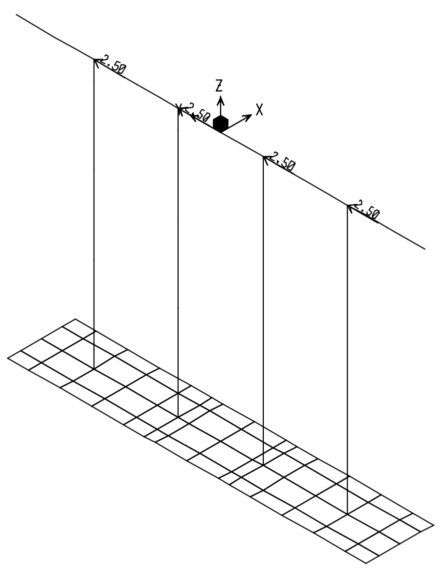
   – Select members to apply hinges
   – Assign > Frame > Hinge
   – Click Add "Auto" hinge
   – Select "Caltrans Flexural Hinge", Select "P-M3" degree of freedom, enter hinge length and check "Drops Load After Point E" as shown below.



6. Verify generated hinge properties, if desired, in the Define > Hinge Properties menu

7. Define load cases

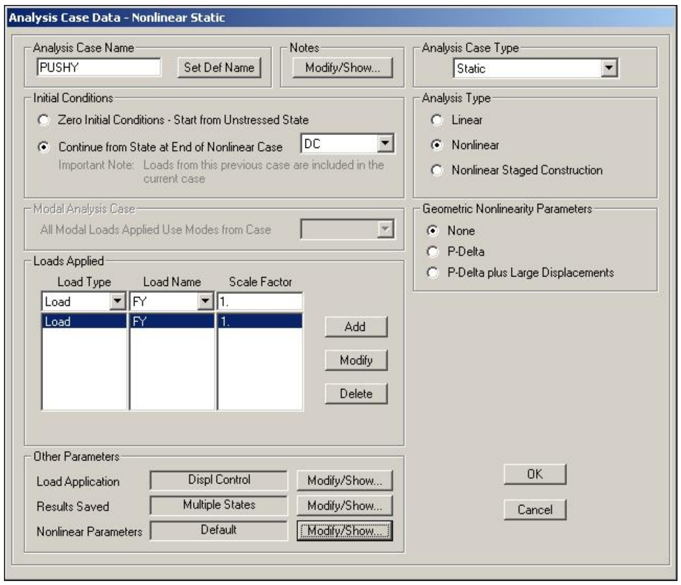
   – Dead load (DC)
   – Unit lateral load at top of columns (FY)



8. Define analysis cases

   – Select dead load case (DC)
   – Click "Modify/Show Case"
   – Select "Nonlinear"

   – Select "Add New Case" and complete form as shown. Monitor deflection up to 30"



9. Run analysis

SHEET 4-26 OF

JOB TITLE **BENT DESIGN** ORIGINATOR Bob Matthews DATE 10/19/2007

JOB No. CALCULATION No. REVIEWER DATE

10. The SAP2000 results are shown below

Model has been checked for input, equilibrium of forces and reasonableness of output.

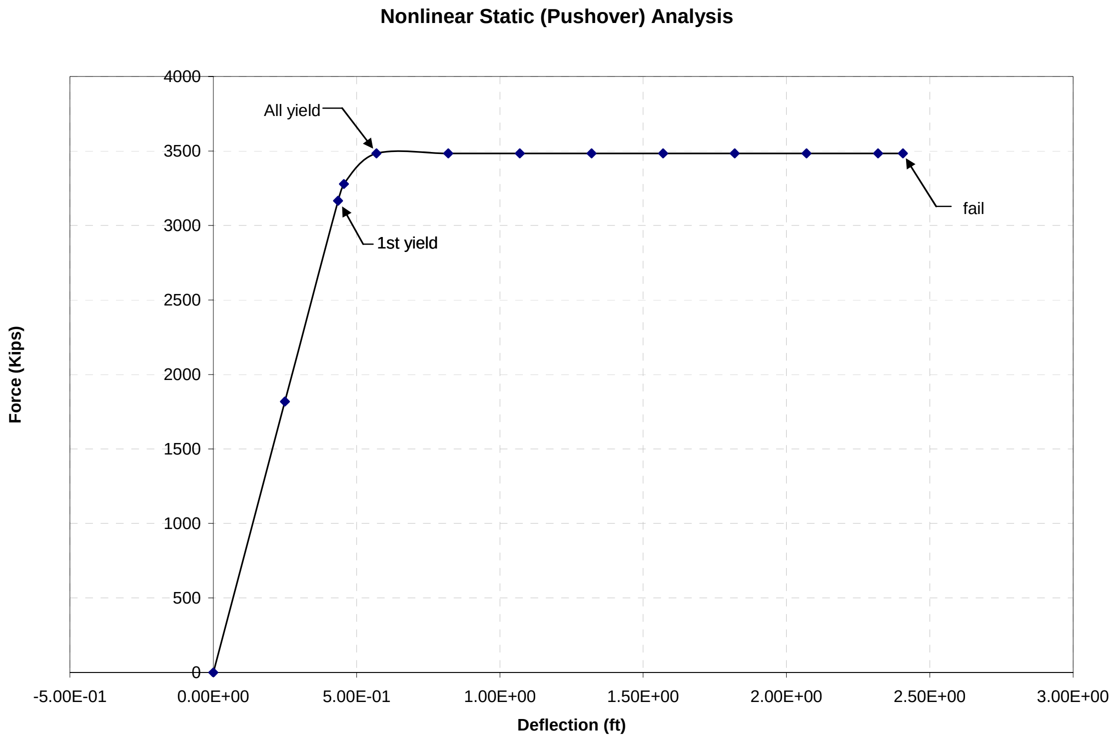


TABLE: Pushover Curve - PUSHY

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD
	ft	Kip		(YIELD)				(FAIL)
0	-1.32E-06	0	4	0	0	0	0	0
1	0.249999	1818.679	4	0	0	0	0	0
2	0.435195	3165.93	3	1	0	0	0	0
3	0.455533	3277.58	1	3	0	0	0	0
4	0.569529	3483.727	0	4	0	0	0	0
5	0.819529	3483.728	0	4	0	0	0	0
6	1.069529	3483.729	0	0	4	0	0	0
7	1.319529	3483.73	0	0	4	0	0	0
8	1.569529	3483.731	0	0	0	4	0	0
9	1.819529	3483.732	0	0	0	4	0	0
10	2.069529	3483.733	0	0	0	1	3	0
11	2.319529	3483.734	0	0	0	0	4	0
12	2.405754	3483.734	0	0	0	0	3	1
13	2.405758	3483.734	0	0	0	0	3	0
14	2.499999	3471.54	0	0	0	0	3	0

Ductility = 2.406 / 0.435 = 5.5

- Bent cap

The critical loads on the bent cap are shown below. A factor of 1.2 is applied to loads on the capacity protected members in accordance with SDC 4.3.1.

CASE	ELEMENT	SHEAR (kips)	MOMENT-3 (k-ft)	COMMENT
PUSHY	14	664	-3322	M-ve
	5	407	1529	M+ve
1.2 X	14	797	-3986	Overstrength loads
	5	488	1835	

- Column

The critical loads on the column are shown below.

CASE	ELEMENT	AXIAL (kips)	SHR-2 (kips)	MOM-3 (k-ft)	COMMENT
PUSHY	404	937	123	3388	Max P
1.2 X	404		148		
PUSHY	104	104	95.8	2636	Min P
1.2 X	104		115		

- Footing

The area elements show high loads concentrated at the outer columns. The code allows the forces to be determined through sections at the face of pins. You can either determine these forces using the pile reactions and manual calculations or use SAP2000 section cut feature to sum the forces. The critical loads on the footing from SAP2000 section cuts are shown below.

CASE	SECTION	V (kips)	M (k-ft)	1.2V/w (kips/ft)	1.2M/w (k-ft/ft)
PUSHY	LONG1	334	430	33.7	43.4
PUSHY	SHORT1	355	1569	53.3	235

- Piles

The critical loads on the piles are shown below.

CASE	JOINT	HORIZ-Y (kips)	AXIAL (kips)	COMMENT
PUSHY	68	-	90.5	Max compression
1.2 X	68		109	
PUSHY	68	-	-150	Max tension
1.2 X	68		-180	
PUSHY	138	-19.2	-	Max shear
1.2 X	138	-23.0		

SECTION 5.0 DESIGN

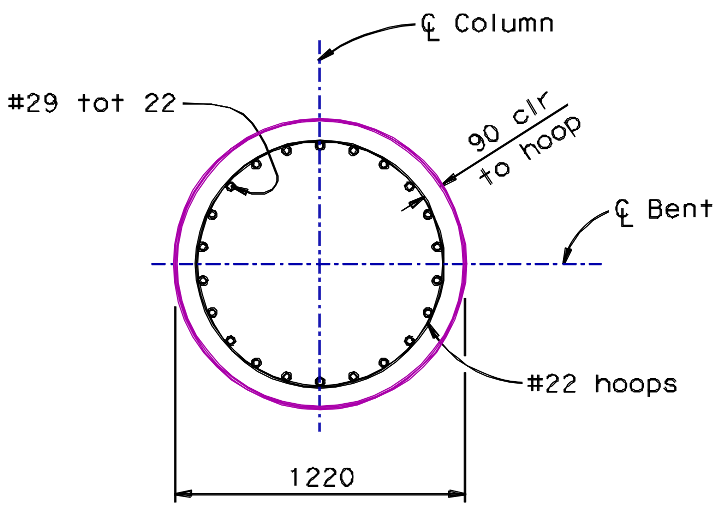
- Caltrans SDC 3.2.1 allows use of expected material properties (except for shear) to determine the capacity of members for seismic loads and Caltrans Amendment 5.5.5 specifies use of a resistance factor of 1.0 for seismic loads. However, nominal material properties with ϕ = 1.0 will be conservatively utilized herein for simplicity.

5.1 BENT FRAME

- Caltrans SDC has requirements for frame displacement capacity and ductility to resist seismic loads

ITEM	DEMAND	CAPACITY	SDC REFERENCE
Displacement X	24.0	25.0	4.1
Displacement Y	20.7	28.9	4.1
Target ductility X	5.0	5.4	2.2.4
Target ductility Y	5.0	5.5	2.2.4

5.2 COLUMN



- Column axial-moment capacity for non-seismic loads is based on LRFD 5.7.4

  Use CONSEC program to analyze column for non-seismic loads

  $P_{SRT\text{-}IIA}$ = 1379 kips
  $M_{SRT\text{-}IIA} = ((481)^2+(174)^2)^{1/2}$ = 512 k-ft

  $M_{DC} = ((96.1)^2+(178)^2)^{1/2}$ = 202 k-ft
  β_d = 202/512 = 0.39 (LRFD 5.7.4.3)
  K = 2.0 for pinned support at footing (LRFD C4.6.2.5)

```
* * * * * * * * * * * * * * * * * * * * * * * * * * *
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*           P R O G R A M   C O N S E C             *
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*                 INPUT DATA ECHO                   *
*                                                   *
* * * * * * * * * * * * * * * * * * * * * * * * * * *
(Version  1.4)                    10/19/07, 1:16 pm

Input file  = nonseismic.in
Output file = nonseismic.out

D E S I G N   C R I T E R I A
=====================================================

Design Criteria = AASHTO LRFD (2004)
Units           = English (inches, pounds)

MATERIAL STRENGTH:

     Concrete compressive strength          = 4000
     Concrete modulus of elasticity         = 3640000
     Reinforcing yield strength             = 60000
     Reinforcing modulus of elasticity      = 2.9E+07

STRENGTH REDUCTION FACTORS:

     Tension and flexure     = 0.9
     Compression and flexure = 0.75
     Shear and torsion       = 0.9

STRESS BLOCK:

     Ratio of average concrete strength  = 0.85
     Ratio of depth of compression block = 0.85
     Maximum concrete strain             = 0.003

RESISTANCE FACTORS:

     Concrete         = 1
     Reinforcing      = 1

C O N C R E T E   C O N F I G U R A T I O N
=====================================================


1 CIRCULAR SECTIONS:

    Ycenter   Radius    Add

     24        24        1
```

```
R E I N F O R C I N G   C O N F I G U R A T I O N
=====================================================


1 REINFORCING ARCS:

    Ycenter    Radius     Abeg       Atot       Nbar       Abar

     24         19         0          343.6364   22         1

L O A D   C O N D I T I O N S
=====================================================
(Units = K-ft)

1 LOAD CONDITIONS:

    No         Axial      Moment     Shear      Torsion

     1         1379.0     512.0      0.0        0.0

M E M B E R   P R O P E R T I E S
=====================================================

MEMBER PROPERTIES:

    Member length                      = 330
    Effective length factor            = 2

BOUNDARY CONDITIONS:

    Condition            End i      End j

    Shear restraint       1          1
    Moment restraint      1          0

S L E N D E R N E S S
=====================================================
(Moments in K-ft)

Moment magnification per design criteria

1 LOAD CONDITIONS:

    No         M1ns       M2ns       M1s        M2s

     1         512.0      0.0        0.0        0.0

    No         BetaD      BetaDs     Tloads

     1          0.39       0          0
```

```
   No        di        dj        ri        rj

    1         0         0         0         0

* * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                   *
*           P R O G R A M   C O N S E C             *
*                                                   *
*                   OUTPUT DATA                     *
*                                                   *
* * * * * * * * * * * * * * * * * * * * * * * * * * *

S E C T I O N   P R O P E R T I E S
=====================================================

GROSS CONCRETE SECTION:

     Area    = 1.8096E+03
     Ybar    = 2.4000E+01
     Io      = 2.6058E+05 About concrete CG

REINFORCING STEEL:

     Area    = 2.2000E+01
     Ybar    = 2.4000E+01
     Io      = 3.9710E+03 About reinf CG

TRANSFORMED CONCRETE SECTION:

     Area    = 1.9628E+03
     Ybar    = 2.4000E+01
     Inertia = 2.8824E+05

I N T E R A C T I O N   D I A G R A M
=====================================================
Moments about centroid of gross concrete section
(Units = K-ft)

c             Mn             Pn
              0.0            -1320.0        Maximum tension
2.35          166.4          -1232.4
4.70          665.0          -928.5
7.05          1255.4         -533.6
9.40          1786.1         -137.8
10.25         1954.3         5.7            Pure bending
11.75         2255.7         258.0
14.10         2674.3         664.8
16.45         3021.1         1082.7
18.80         3294.0         1504.1
20.70         3463.9         1848.1         Load condition no 1
21.15         3494.9         1932.8
23.50         3606.2         2362.5
```

```
25.35         3628.5         2718.4         Balanced strain
25.85         3617.1         2820.9
28.20         3542.2         3287.9
30.55         3428.9         3726.4
32.90         3270.9         4149.3
35.25         3073.4         4546.8
37.60         2832.5         4933.5
39.95         2554.0         5301.8
42.30         2248.2         5643.6
44.65         1909.7         5969.8
47.00         1560.4         6262.4
              0.0            6288.1         Maximum compression

S L E N D E R N E S S   E F F E C T S
=====================================================

SLENDERNESS PER AASHTO LRFD (2004):

     LOAD CONDITION 1:

          Pc          = 4969.3

          r           = 12.000
          kl/r        = 55.0
          kl/r limit  = 34.0

          Equivalent moment factor, Cm   = 0.600
          Non-sway magnification factor  = 1.000
          Sway magnification factor      = 1.000
          Magnified moment               = 512.0

S E C T I O N   A N A L Y S I S
=====================================================
(Units = K-ft)

LOAD CONDITION NO 1

     For Axial load      = 1386.1
     Reduction factor    = 0.750
     Moment capacity     = 2597.9
     Initial moment      = 512.0
     Magnified moment    = 512.0 < 2598 Okay
```

- Check column slenderness effects for seismic loads (SDC 4.2)

Slenderness effects can be ignored if $P\Delta < 0.2M_p$

$P_d\Delta$ = 467 x 20.7 / 12 = 806 k-ft
P_d = Axial compression load in column due to dead load (no overturning effects)
0.2 M_p = 0.2 x 2991 = 598 k-ft < 806 k-ft, $\therefore$ slenderness effects cannot be ignored

The effects of $P\Delta$ on the demand displacement can only be captured accurately using nonlinear time-history analysis. The effects of $P\Delta$ on the capacity can be included in the nonlinear static (pushover) analysis with SAP2000. For actual design, the column reinforcing would be increased to reduce the deflection and increase the plastic moment capacity. This would require an iteration to the global and local seismic analysis. For the purposes of this example problem, $P\Delta$ effects will not be included.

- Column transverse reinforcing requirements for non-seismic loads are shown in LRFD 5.7.4.6, 5.8 & 5.10.6

  – Minimum ratio of hoop reinforcement (LRFD 5.7.4.6)

$p_s = 4 \times A_b / (D_r \times s) > 0.45[A_g/A_c - 1](f'_c/f_{yh})$

A_c = Area of core measured to outside diameter of hoop

$p_s = 4 \times 0.6 / (40 \times 3) = 0.02 > 0.45[(48)^2/(41)^2 - 1](4 / 60) = 0.011$ Okay

  – Manual calculations for shear strength use (LRFD 5.8, C5.8.2.9)

Manual calculations using modified compression field theory are required to review circular column transverse reinforcing for non-seismic loads. Response 2000 is capable of performing this analysis, but this program is not widely used.

effective width $b_v = D$

$d_v = M_n / (A_s f_y)$ where M_n ignores axial load and A_s is one-half longitudinal reinforcement
or
$d_v = 0.9(D / 2 + D_r / \pi)$

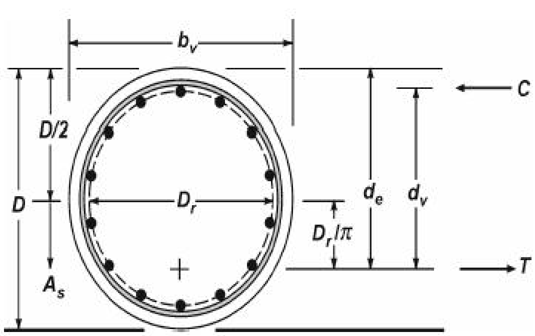


Figure C5.8.2.9-2 Illustration of Terms b_v, d_v and d_e for Circular Sections.

Refer to the course notes on LRFD Shear Design for details of manual calculations.

– Detailing requirements are shown in LRFD 5.10.6

1. Minimum bar size is #3
2. Clear spacing is 1 inch or 1.33 x d_b
3. Splice requirements for spirals

- Column transverse reinforcing requirements for seismic loads (ductile columns) are shown in LRFD 5.10.11 and Caltrans SDC 3.6 & 3.8

  Since this column is in a high seismic zone, LRFD and SDC require similar transverse reinforcing requirements for ductile columns. Namely, that the column shear is determined from the column overstrength moment. SDC requirements are shown below.

– Minimum ratio of hoop reinforcement (SDC 3.6.5.2)

$A_v > 0.025$ (D' x s / f_{yh})
$A_v = 0.025 \times 40 \times 3 / 60 = 0.05 < 0.6$ Okay

– Manual calculations for shear strength (SDC 3.6)

Typically review max and min axial load with associated shear
Check overstrength shear V_o = 148 kips with P_c = 937 kips

$V_s = \pi \times A_b \times f_{yh} \times D' / 2s = 3.1416 \times 0.6 \times 60 \times 40 / (2 \times 3) = 754$ kips
$V_s < 8 \times (f'_c)^{1/2}\ 0.8 \times A_g = 8 \times (4000)^{1/2} \times 0.8 \times 1810 / 1000 = 733$ kips (controls)

$V_c \Rightarrow F_1 \times F_2 \times 2(f'_c)^{1/2} \times 0.8 \times A_g < 4(f'_c)^{1/2} \times 0.8 \times A_g$
$F_1 \Rightarrow 0.3 < p_s \times f_{yh} / 0.15 + 3.67 - \mu_d < 3$
$F_2 \Rightarrow 1 + P_c / (2000 \times A_g) < 1.5$

μ_d = Ductility demand ratio (x) = 24.0 / 4.6 = 5.2
p_s = Ratio of hoop reinforcement = 4 x 0.6 / (40 x 3) = 0.02
$V_c = 3 \times 1.26 \times (4000)^{1/2} \times 0.8 \times 1810 / 1000 = 346$ kips

$\phi(V_c + V_s) = 0.85(1079) = 917 > 148$ Okay

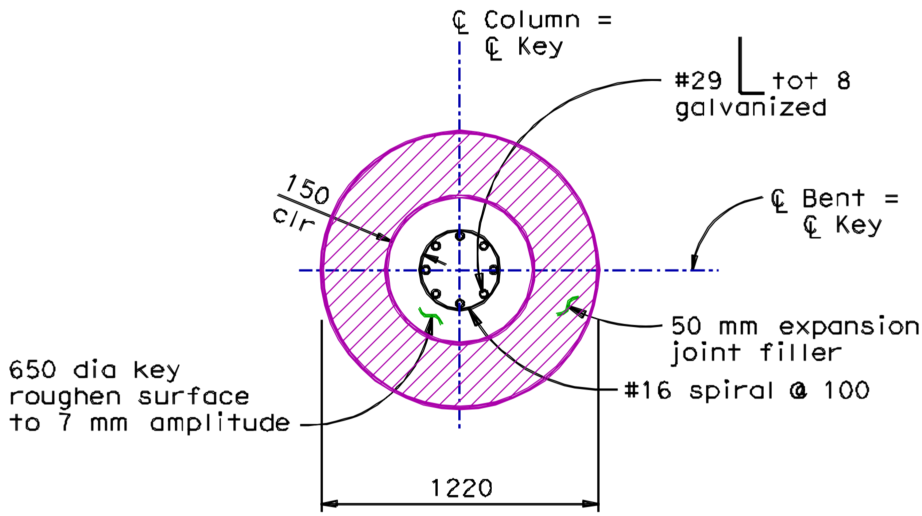
– Detailing requirements are shown in SDC 8.2.5

Spacing < D/5 or 6 d_b or 8"
6 x 0.875 = 5.25 > 3 Okay

- Column local ductility requirements (SDC 3.1.4)

  Local ductility = 5.4 > 3.0 Okay

- Column pin requirements (LRFD 5.8.4)



Shear friction is used for load transfer

V = 148 kips with P_c = 937 kips
V = 115 kips with P_c = 104 kips

The nominal shear resistance of the interface plane shall be taken as:

$$V_n = cA_{cv} + \mu\left[A_{vf} f_y + P_c\right] \qquad (5.8.4.1\text{-}1)$$

The nominal shear resistance, V_n, used in the design shall not be greater than the lesser of:

$$V_n \le 0.2 f'_c A_{cv}\text{, or} \qquad (5.8.4.1\text{-}2)$$

$$V_n \le 0.8 A_{cv} \qquad (5.8.4.1\text{-}3)$$

Use key diameter = 25.6" => A_{cv} = 515 in^2 > 148/0.8 = 185 Okay

c = 0.1 ksi
μ = 1.0
ϕ = 0.9 (LRFD 5.5.4.2.1) for non-seismic loads
ϕ = 1.0 (Caltrans Amendments 5.5.5) for seismic loads

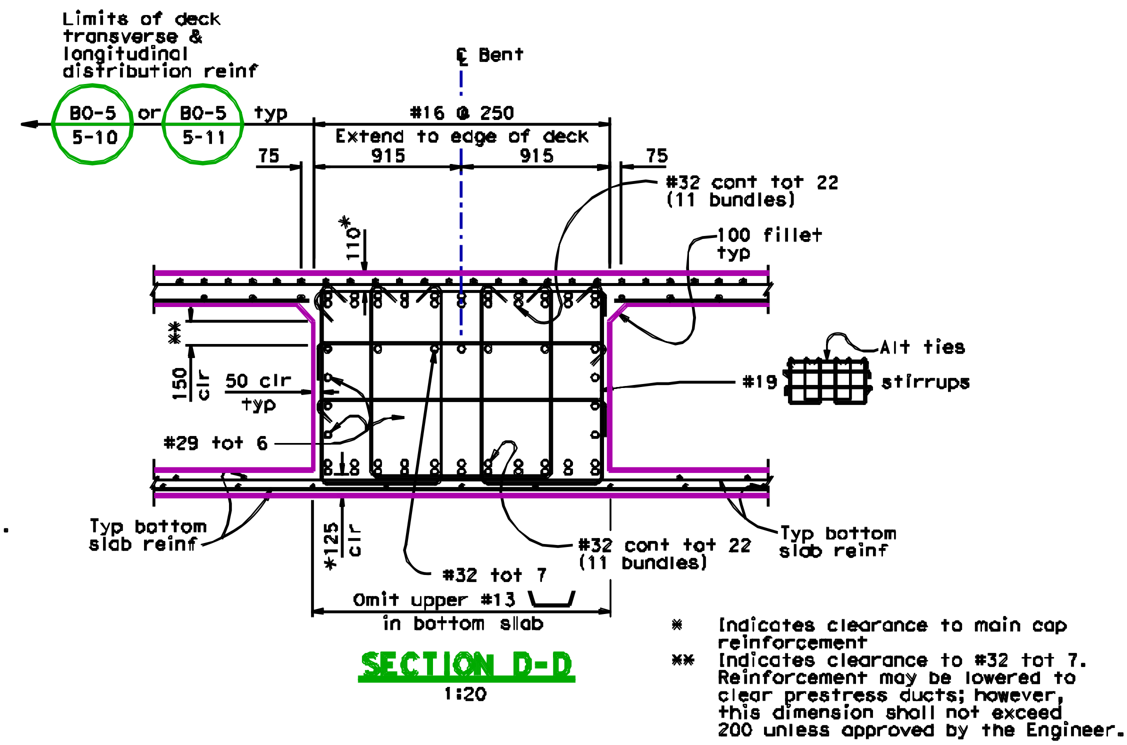
ϕV_n = 0.1 x 515 + 1.0[8 x 60 + 104] = 636 kips > 104 Okay

5.3 BENT CAP

- Several programs are available to perform bent cap design as shown below.

Program	Description
SAP2000	General finite element program concrete design features
RCPIER	Program designed specifically for bent analysis and design. This program does not include the Caltrans amendments to LRFD or the Caltrans SDC requirements, but still may be used to determine the effects of most of the bent loadings.
REBEAM	Reinforced concrete beam design program

REBEAM will be used for bent cap design.



- Bent cap capacity for non-seismic loads is based on LRFD 5.

Check negative moment and shear capability and crack control

V_f = 732 kips
M_f = 2778 k-ft
M_s = 1271 k-ft

```
* * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                   *
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*                   OUTPUT DATA                     *
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(Version  5.1)                   10/22/07, 2:44 pm

Output file = nonseismic.out

Shear and moment review problem

D E S I G N   C R I T E R I A
====================================================

Code  = AASHTO LRFD (2004)
Units = English (pounds, inches - shear and moments in kip-ft)

STRENGTH REDUCTION AND RESISTANCE FACTORS:

     Flexure reduction factor            = 0.90
     Shear reduction factor              = 0.90
     Concrete resistance factor          = 1.00
     Reinforcing resistance factor       = 1.00

STRESS BLOCK:

     Ratio of average concrete strength  = 0.8500
     Ratio of depth of compression block = 0.8500
     Maximum concrete strain             = 0.0030

M A T E R I A L   P R O P E R T I E S
====================================================

     Concrete compressive strength       = 4000
     Concrete modulus of elasticity      = 3.6400E+06
     Concrete modulus of rupture         = 480
     Reinforcing yield strength          = 60000
     Reinforcing modulus of elasticity   = 2.9000E+07
     Modular ratio                       = 8
     Maximum aggregate size              = 1.000
```

		SHEET	5-11	OF	
JOB TITLE	BENT DESIGN	ORIGINATOR	Bob Matthews	DATE	10/31/2007
JOB No.	CALCULATION No.	REVIEWER		DATE	

```
R E I N F O R C I N G   S T E E L
====================================================

     Tensile reinf area          = 27.94
     Depth to tensile reinf      = 48.2
     Compressive reinf area      = 27.94
     Depth to compressive reinf = 6.4
     Shear reinf area            = 2.64
     Shear reinf spacing         = 6

D E S I G N   L O A D S
====================================================

     Factored (ultimate) moment  = 2778
     Maximum service load moment = 1271
     Minimum service load moment = 0
     Factored (ultimate) shear   = 732

S E C T I O N   P R O P E R T I E S
====================================================

RECTANGULAR SECTION:

     Width  = 72
     Height = 54

PROPERTIES:

     Gross moment of inertia     = 9.4478E+05
     Gross section modulus       = 3.4992E+04
     Distance to neutral axis    = 2.7000E+01
     Cracked moment of inertia   = 3.3802E+05
     Effective moment of inertia = 9.4478E+05

M O M E N T   R E V I E W   C A L C U L A T I O N S
====================================================

MINIMUM REINFORCING:

     1.2 * Cracking moment      = 1.6796E+03
     Design moment              = 2.7780E+03

MAXIMUM REINFORCING:

     c/d                        = 1.4951E-01
     Maximum c/d                = 4.2000E-01

MOMENT CAPABILITY:

     Ultimate moment capability  = 5.6071E+03 > 2778 Okay
     Stress block depth          = 6.1252E+00
     Stress in compression steel = 9.7329E+03
```

```
SERVICE LOAD STRESS:

     Maximum steel stress    = 1.2588E+04
     Minimum steel stress    = 0.0000E+00
     Maximum concrete stress = 5.4316E+02
     Minimum concrete stress = 0.0000E+00

CRACK CONTROL:

     Concrete cover            = 2.5
     Effective tension area    = 835
     Exposure factor           = 170000
     Maximum steel stress      = 1.2588E+04
     Allowable cracking stress = 1.3336E+04 > 12588 Okay

S H E A R   R E V I E W   C A L C U L A T I O N S
====================================================

CONCRETE SHEAR CAPABILITY:

     Effective shear depth      = 4.4596E+01
     Concrete shear strength    = 4.0757E+02
        Longitudinal strain     = 7.6762E-04
        Theta                   = 3.6400E+01
        Beta                    = 2.2300E+00

SHEAR REINFORCING:

     Min shear reinf area       = 4.5537E-01
     Max shear reinf spacing    = 2.4000E+01

     Shear reinf strength       = 1.4372E+03
     Ultimate shear capability  = 1.8448E+03 > 732 Okay
     Maximum shear capability   = 2.8898E+03

MINIMUM LONGITUDINAL TENSILE REINFORCING:

     Minimum reinf area         = 2.3036E+01
```

- Bent cap capacity for seismic loads is based on LRFD 5 and Caltrans Amendments

ϕ = 1.0 (Caltrans Amendments 5.5.5)

Check negative moment and shear capability

V_f = 797 kips
M_f = 3986 k-ft

```
* * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                   *
*           P R O G R A M   R E B E A M             *
*                                                   *
*                   OUTPUT DATA                     *
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(Version  5.1)                   10/22/07, 2:45 pm

Output file = seismic.out

Shear and moment review problem

D E S I G N   C R I T E R I A
====================================================

Code  = AASHTO LRFD (2004)
Units = English (pounds, inches - shear and moments in kip-ft)

STRENGTH REDUCTION AND RESISTANCE FACTORS:

     Flexure reduction factor            = 1.00
     Shear reduction factor              = 1.00
     Concrete resistance factor          = 1.00
     Reinforcing resistance factor       = 1.00

STRESS BLOCK:

     Ratio of average concrete strength  = 0.8500
     Ratio of depth of compression block = 0.8500
     Maximum concrete strain             = 0.0030

M A T E R I A L   P R O P E R T I E S
====================================================

     Concrete compressive strength       = 4000
     Concrete modulus of elasticity      = 3.6400E+06
     Concrete modulus of rupture         = 480
     Reinforcing yield strength          = 60000
     Reinforcing modulus of elasticity   = 2.9000E+07
     Modular ratio                       = 8
     Maximum aggregate size              = 1.000
```

```
R E I N F O R C I N G   S T E E L
====================================================

     Tensile reinf area          = 27.94
     Depth to tensile reinf      = 48.2
     Compressive reinf area      = 27.94
     Depth to compressive reinf = 6.4
     Shear reinf area            = 2.64
     Shear reinf spacing         = 6

D E S I G N   L O A D S
====================================================

     Factored (ultimate) moment  = 3986
     Maximum service load moment = 0
     Minimum service load moment = 0
     Factored (ultimate) shear   = 797

S E C T I O N   P R O P E R T I E S
====================================================

RECTANGULAR SECTION:

     Width  = 72
     Height = 54

PROPERTIES:

     Gross moment of inertia     = 9.4478E+05
     Gross section modulus       = 3.4992E+04
     Distance to neutral axis    = 2.7000E+01
     Cracked moment of inertia   = 3.3802E+05

M O M E N T   R E V I E W   C A L C U L A T I O N S
====================================================

MINIMUM REINFORCING:

     1.2 * Cracking moment      = 1.6796E+03
     Design moment              = 3.9860E+03

MAXIMUM REINFORCING:

     c/d                        = 1.4951E-01
     Maximum c/d                = 4.2000E-01

MOMENT CAPABILITY:

     Ultimate moment capability  = 6.2301E+03 > 3986 Okay
     Stress block depth          = 6.1252E+00
     Stress in compression steel = 9.7329E+03
```

```
S H E A R   R E V I E W   C A L C U L A T I O N S
====================================================

CONCRETE SHEAR CAPABILITY:

     Effective shear depth      = 4.4596E+01
     Concrete shear strength    = 4.5286E+02
        Longitudinal strain     = 9.9540E-04
        Theta                   = 3.6400E+01
        Beta                    = 2.2300E+00

SHEAR REINFORCING:

     Min shear reinf area       = 4.5537E-01
     Max shear reinf spacing    = 2.4000E+01

     Shear reinf strength       = 1.5969E+03
     Ultimate shear capability  = 2.0498E+03 > 797 Okay
     Maximum shear capability   = 3.2109E+03

MINIMUM LONGITUDINAL TENSILE REINFORCING:

     Minimum reinf area         = 2.6885E+01
```

- Check column reinforcement development in bent cap (LRFD 5.11 & 5.10.11.4.3 and SDC 8.2.1)

$L_{db} > 1.25A_b f_y/(f'_c)^{1/2} = 1.25 \times 1.0 \times 60 / (4)^{1/2} = 37.5$" (LRFD 5.11.2.1)
Can reduce by 0.75 enclosed with spirals => 28.1"
Increase by 25% for high seismic region => 35.1" (LRFD 5.10.11.4.3)

Embed column reinforcement as close to far end of cap as practical, but not less than $24d_b$ (SDC 8.2.1)

Depth to bottom of top cap reinforcement = 54 - 4.3 - 2 x 1.4 = 46.9"
24 x 1.128 = 27"

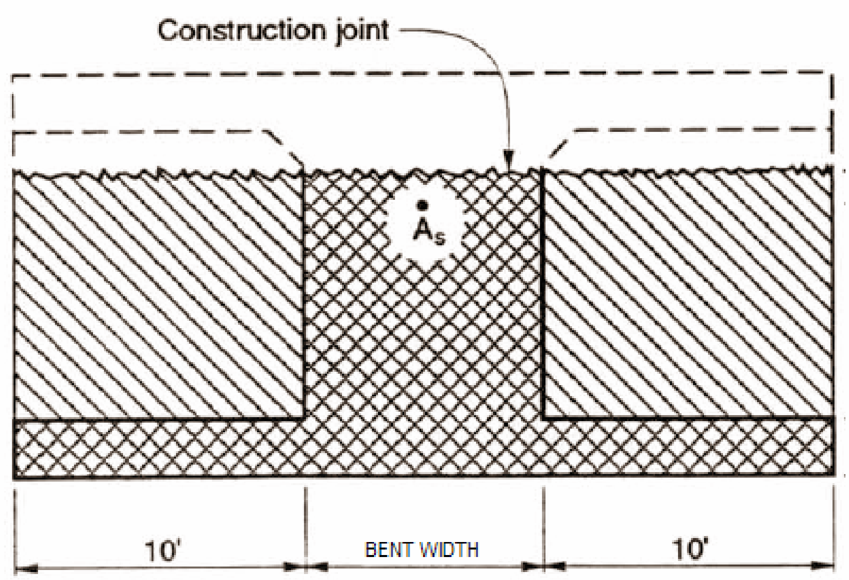
43.3" provided Okay

- Check side face reinforcement (LRFD 5.8.3.4.2)

Crack control reinf for shear $A_s > 0.003 b_v s_x$
0.003 x 72 x 7 = 1.5 in^2 < 2.0 in^2 (2 #9) provided okay

- Construction reinforcement (Caltrans Bridge Design Practice 2.33.0)

It is Caltrans practice to place construction reinforcement below the deck construction joint. This reinforcement is designed to carry the dead load of the cap and 10' of the superstructure on each side of the cap using a load factor of 1.3 with f'c = 2500 psi.



Depth to construction joint = 42.3" (3.52')
Depth to construction reinforcement = 35.5"
Bottom slab thickness = 0.51'
Bottom slab width = 39.5'
Total girder thickness = 6.3'

Girder reaction = 0.15 x 20 (6.3 x (3.52 - .51) + 39.5 x 0.51) = 117 kips
$M = 1.3(117 \times 2.3 + 0.15 \times 6 \times 3.52\ (2.3)^2/2 + 0.15 \times 6 \times 5 \times 1 \times 4.8) = 389$ k-ft

Use REBEAM to check (7) #10 bars, $A_s = 8.89\ in^2$

```
* * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                   *
*           P R O G R A M   R E B E A M             *
*                                                   *
*                   OUTPUT DATA                     *
*                                                   *
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(Version  5)                      10/19/07, 4:57 pm

Output file = construction.out

Moment review problem

D E S I G N   C R I T E R I A
====================================================

Code  = AASHTO LRFD (2004)
Units = English (pounds, inches - shear and moments in kip-ft)

STRENGTH REDUCTION AND RESISTANCE FACTORS:

     Flexure reduction factor            = 0.90
     Shear reduction factor              = 0.90
     Concrete resistance factor          = 1.00
     Reinforcing resistance factor       = 1.00

STRESS BLOCK:

     Ratio of average concrete strength  = 0.8500
     Ratio of depth of compression block = 0.8500
     Maximum concrete strain             = 0.0030

M A T E R I A L   P R O P E R T I E S
====================================================

     Concrete compressive strength       = 2500
     Concrete modulus of elasticity      = 2.8777E+06
     Concrete modulus of rupture         = 379.5
     Reinforcing yield strength          = 60000
     Reinforcing modulus of elasticity   = 2.9000E+07
     Modular ratio                       = 10
     Maximum aggregate size              = 1.000
```

```
R E I N F O R C I N G   S T E E L
====================================================

     Tensile reinf area          = 8.89
     Depth to tensile reinf      = 35.5
     Compressive reinf area      = 0
     Depth to compressive reinf = 0
     Shear reinf area            = 0
     Shear reinf spacing         = 0

D E S I G N   L O A D S
====================================================

     Factored (ultimate) moment  = 389
     Maximum service load moment = 0
     Minimum service load moment = 0
     Factored (ultimate) shear   = 0

S E C T I O N   P R O P E R T I E S
====================================================

RECTANGULAR SECTION:

     Width  = 72
     Height = 42.3

PROPERTIES:

     Gross moment of inertia     = 4.5412E+05
     Gross section modulus       = 2.1471E+04
     Distance to neutral axis    = 2.1150E+01
     Cracked moment of inertia   = 7.9489E+04

M O M E N T   R E V I E W   C A L C U L A T I O N S
====================================================

MINIMUM REINFORCING:

     1.2 * Cracking moment      = 8.1484E+02
     Design moment              = 5.1867E+02

MAXIMUM REINFORCING:

     c/d                        = 1.1554E-01
     Maximum c/d                = 4.2000E-01

MOMENT CAPABILITY:

     Ultimate moment capability  = 1.3504E+03  > 519 Okay
     Stress block depth          = 3.4863E+00
```

- Check development of main reinforcement (LRFD 5.11.2.1)

$L_{db} > 1.25A_b f_y/(f'_c)^{1/2} = 1.25 \times 1.27 \times 60 / (4)^{1/2} = 48"$
Increase by 40% for top reinforcement => 67"

Bottom reinforcement must be developed at least 48" from inside face of column for seismic opening moment. Actual length = 75" > 48" Okay

Top reinforcement must be developed at least 67" from outside face of column for seismic closing moment. Actual length = 84" > 67" Okay

- Check joint shear (SDC 7.4)

Calculate principal stresses in the joint

$$p_t = \frac{(f_h + f_v)}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2}$$

$$p_c = \frac{(f_h + f_v)}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2}$$

$$v_{jv} = T_c / A_{jv}$$

$$A_{jv} = l_{ac} \times B_{cap}$$

$$f_v = \frac{P_c}{A_{jh}}$$

$$A_{jh} = (D_c + D_s) \times B_{cap}$$

$$f_h = \frac{P_b}{B_{cap} \times D_s}$$

Where:

A_{jh} = The effective horizontal joint area
A_{jv} = The effective vertical joint area
B_{cap} = Bent cap width
D_c = Cross–sectional dimension of column in the direction of bending
D_s = Depth of superstructure at the bent cap
l_{ac} = Length of column reinforcement embedded into the bent cap
P_c = The column axial force including the effects of overturning
P_b = The beam axial force at the center of the joint including prestressing
T_c = The column tensile force defined as M_o / h, where h is the distance from c.g. of tensile force to c.g. of compressive force on the section, or alternatively T_c may be obtained from the moment curvature analysis of the cross section.

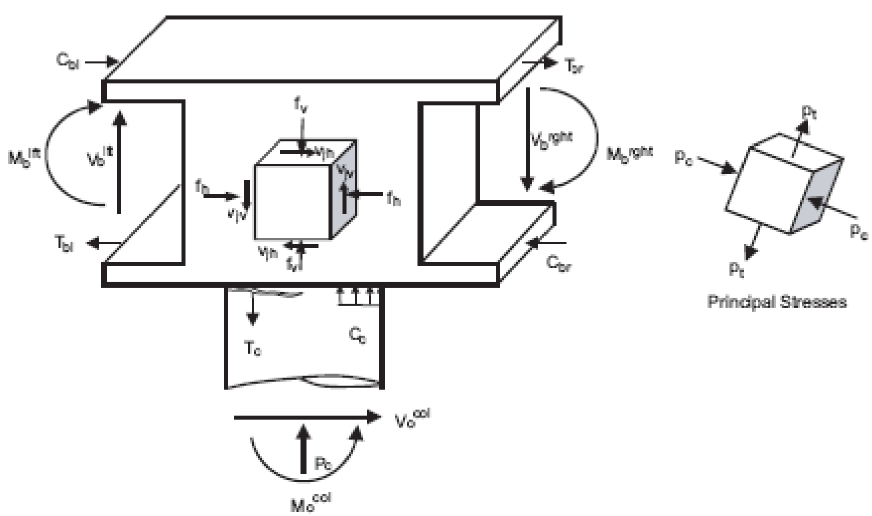
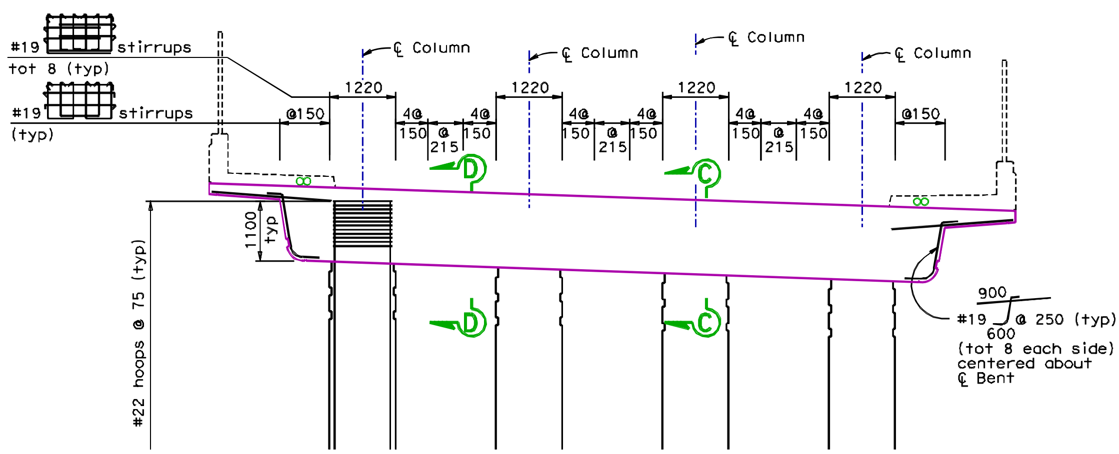


Figure 7.6 Joint Shear Stresses in T Joints

$B_{cap} = 72$
$D_c = 48$
$D_s = 54$
$l_{ac} = 43.3$
$A_{jh} = (48 + 54) \times 72 = 7344$
$A_{jv} = 43.3 \times 72 = 3118$
$P_c = 937$
$P_b = 0$
$T_c = 1142$ (from moment-curvature analysis)

$f_v = 937000 / 7344 = 128$
$f_h = 0$
$v_{jv} = 1142000 / 3118 = 366$

$p_t = -308$ psi $> 3.5(4000)^{1/2} = 221$ psi, $\therefore$ special joint reinforcing must be provided (SDC 7.4.4.2)
$p_t = -308$ psi $< 12(4000)^{1/2} = 759$ psi, $\therefore$ joint size is okay (SDC 7.4.2)
$p_c = 436$ psi $< 0.25(4000) = 1000$ psi, $\therefore$ joint size is okay (SDC 7.4.2)



ELEVATION

A) Vertical stirrups

Required $A_s^{jv} = 0.2 \times A_{st} = 0.2 \times 22 \times 1.0 = 4.4\ in^2$
Provided $A_s^{jv} = 4 \times 6 \times 0.44 = 10.56\ in^2$ Okay

B) Horizontal stirrups

Required $A_s^{jv} = 0.2 \times A_{st} = 0.1 \times 22 \times 1.0 = 2.2\ in^2$
Provided $A_s^{jv} = 4 \times 2 \times 0.44 = 3.52\ in^2$ Okay

C) Horizontal side reinforcement

Required $A_s^{sf} = 0.1 \times A_{cap}{}^{top}{}_t = 0.1 \times 22 \times 1.27 = 2.8\ in^2$
Provided $A_s^{sf} = 8 \times 1.0 = 8\ in^2$ Okay

D) J-Dowels (not required for skew < 20 degrees)

E) Transverse reinforcement

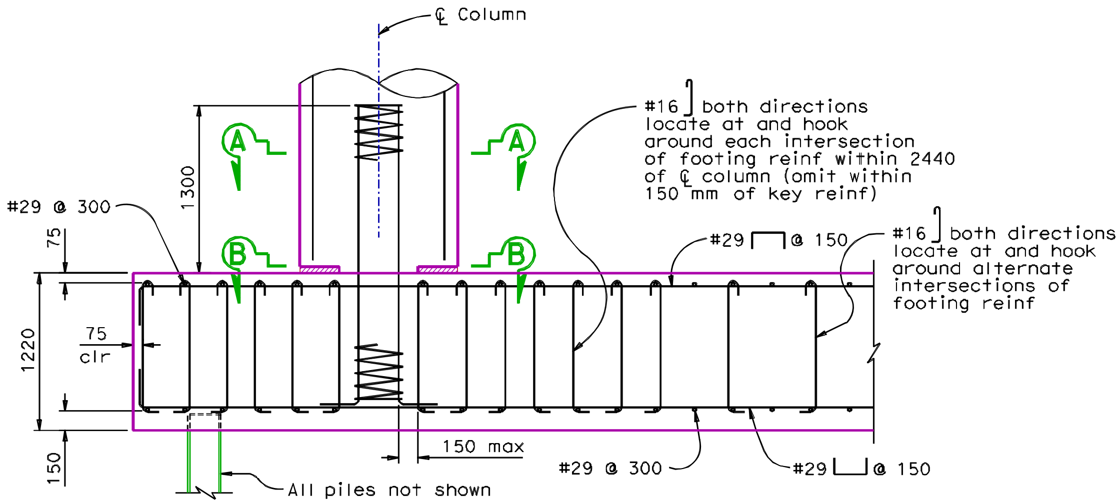
Required $\rho_s = 0.4 \times A_{st} / l_{ac}^2 = 0.4 \times 22 / (43.3)^2 = 0.0047$
Provided $\rho_s = 0.02$ Okay

5.4 FOOTING

- Several programs are available to perform footing design as shown below.

Program	Description
SAP2000	General finite element program concrete design features
RCPIER	Program designed specifically for bent analysis and design. This program does not include the Caltrans amendments to LRFD or the Caltrans SDC requirements, but still may be used to determine the effects of most of the bent loadings. The footing analysis is done on a rigid model separate from the rest of the bent model.
REBEAM	Reinforced concrete beam design program

REBEAM will be used for footing design.



FOOTING DETAIL

- Footing capacity for non-seismic loads is based on LRFD 5.

Check moment and shear capability for bottom reinforcement in the long direction. Nonseismic loads control in this direction.

V_f = 58.2 kips/ft
M_f = 76.8 k-ft/ft

```
* * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                   *
*           P R O G R A M   R E B E A M             *
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*                   OUTPUT DATA                     *
*                                                   *
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(Version  5.1)                    10/22/07, 2:20 pm

Output file = nonseismic.out

Shear and moment review problem

D E S I G N   C R I T E R I A
====================================================

Code  = AASHTO LRFD (2004)
Units = English (pounds, inches - shear and moments in kip-ft)

STRENGTH REDUCTION AND RESISTANCE FACTORS:

     Flexure reduction factor             = 0.90
     Shear reduction factor               = 0.90
     Concrete resistance factor           = 1.00
     Reinforcing resistance factor        = 1.00

STRESS BLOCK:

     Ratio of average concrete strength  = 0.8500
     Ratio of depth of compression block = 0.8500
     Maximum concrete strain             = 0.0030

M A T E R I A L   P R O P E R T I E S
====================================================

     Concrete compressive strength       = 4000
     Concrete modulus of elasticity      = 3.6400E+06
     Concrete modulus of rupture         = 480
     Reinforcing yield strength          = 60000
     Reinforcing modulus of elasticity   = 2.9000E+07
     Modular ratio                       = 8
     Maximum aggregate size              = 1.000
```

JOB TITLE **BENT DESIGN** ORIGINATOR Bob Matthews DATE 10/31/2007

JOB No. CALCULATION No. REVIEWER DATE

```
R E I N F O R C I N G   S T E E L
====================================================

     Tensile reinf area          = 1
     Depth to tensile reinf      = 42
     Compressive reinf area      = 0
     Depth to compressive reinf = 0
     Shear reinf area            = 0.31
     Shear reinf spacing         = 6

D E S I G N   L O A D S
====================================================

     Factored (ultimate) moment  = 76.8
     Maximum service load moment = 0
     Minimum service load moment = 0
     Factored (ultimate) shear   = 58.2

S E C T I O N   P R O P E R T I E S
====================================================

RECTANGULAR SECTION:

     Width  = 12
     Height = 48

PROPERTIES:

     Gross moment of inertia     = 1.1059E+05
     Gross section modulus       = 4.6080E+03
     Distance to neutral axis    = 2.4000E+01
     Cracked moment of inertia   = 1.1170E+04

M O M E N T   R E V I E W   C A L C U L A T I O N S
====================================================

MINIMUM REINFORCING:

     1.2 * Cracking moment      = 2.2118E+02
     Design moment              = 1.0240E+02

MAXIMUM REINFORCING:

     c/d                        = 4.1193E-02
     Maximum c/d                = 4.2000E-01

MOMENT CAPABILITY:

     Ultimate moment capability  = 1.8569E+02 > 76.8 Okay
     Stress block depth          = 1.4706E+00
```

```
S H E A R   R E V I E W   C A L C U L A T I O N S
====================================================

CONCRETE SHEAR CAPABILITY:

     Effective shear depth      = 4.1265E+01
     Concrete shear strength    = 6.2855E+01
        Minimum moment          = 2.0013E+02
        Longitudinal strain     = 1.0000E-03
        Theta                   = 3.6400E+01
        Beta                    = 2.2300E+00

SHEAR REINFORCING:

     Min shear reinf area       = 7.5895E-02
     Max shear reinf spacing    = 2.4000E+01

     Shear reinf strength       = 1.5616E+02
     Ultimate shear capability  = 2.1901E+02 > 58.2 Okay
     Maximum shear capability   = 4.4566E+02

MINIMUM LONGITUDINAL TENSILE REINFORCING:

     Minimum reinf area         = 1.1445E+00
```

- Footing capacity for seismic loads is based on LRFD 5 and Caltrans Amendments

  ϕ = 1.0 (Caltrans Amendments 5.5.5)

  Check moment and shear capability for bottom reinforcement in the short direction. Seismic loads control in this direction. Expected material properties will be used since the nominal properties showed negative margin of safety.

  V_f = 53.3 kips/ft
  M_f = 235 k-ft/ft

```
* * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                   *
*           P R O G R A M   R E B E A M             *
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*                   OUTPUT DATA                     *
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(Version  5.1)                   10/22/07, 2:38 pm

Output file = seismic.out

Shear and moment review problem

D E S I G N   C R I T E R I A
====================================================

Code  = AASHTO LRFD (2004)
Units = English (pounds, inches - shear and moments in kip-ft)

STRENGTH REDUCTION AND RESISTANCE FACTORS:

     Flexure reduction factor            = 1.00
     Shear reduction factor              = 1.00
     Concrete resistance factor          = 1.00
     Reinforcing resistance factor       = 1.00

STRESS BLOCK:

     Ratio of average concrete strength  = 0.8500
     Ratio of depth of compression block = 0.7900
     Maximum concrete strain             = 0.0030

M A T E R I A L   P R O P E R T I E S
====================================================

     Concrete compressive strength       = 5200
     Concrete modulus of elasticity      = 4.1502E+06
     Concrete modulus of rupture         = 547.3
     Reinforcing yield strength          = 68000
     Reinforcing modulus of elasticity   = 2.9000E+07
     Modular ratio                       = 7
     Maximum aggregate size              = 1.000
```

```
R E I N F O R C I N G   S T E E L
====================================================

     Tensile reinf area          = 1
     Depth to tensile reinf      = 42
     Compressive reinf area      = 0
     Depth to compressive reinf = 0
     Shear reinf area            = 0.62
     Shear reinf spacing         = 12

D E S I G N   L O A D S
====================================================

     Factored (ultimate) moment  = 235
     Maximum service load moment = 0
     Minimum service load moment = 0
     Factored (ultimate) shear   = 53.3

S E C T I O N   P R O P E R T I E S
====================================================

RECTANGULAR SECTION:

     Width  = 12
     Height = 48

PROPERTIES:

     Gross moment of inertia     = 1.1059E+05
     Gross section modulus       = 4.6080E+03
     Distance to neutral axis    = 2.4000E+01
     Cracked moment of inertia   = 9.9200E+03

M O M E N T   R E V I E W   C A L C U L A T I O N S
====================================================

MINIMUM REINFORCING:

     1.2 * Cracking moment      = 2.5220E+02
     Design moment              = 2.5220E+02

MAXIMUM REINFORCING:

     c/d                        = 3.8639E-02
     Maximum c/d                = 4.2000E-01

MOMENT CAPABILITY:

     Ultimate moment capability  = 2.3437E+02 ≅ 235 Okay
     Stress block depth          = 1.2821E+00
```

```
S H E A R   R E V I E W   C A L C U L A T I O N S
====================================================

CONCRETE SHEAR CAPABILITY:

     Effective shear depth      = 4.1359E+01
     Concrete shear strength    = 7.9810E+01
        Longitudinal strain     = 1.0000E-03
        Theta                   = 3.6400E+01
        Beta                    = 2.2300E+00

SHEAR REINFORCING:

     Min shear reinf area       = 1.5271E-01
     Max shear reinf spacing    = 2.4000E+01

     Shear reinf strength       = 1.9709E+02
     Ultimate shear capability  = 2.7690E+02 > 53.3 Okay
     Maximum shear capability   = 6.4520E+02

MINIMUM LONGITUDINAL TENSILE REINFORCING:

     Minimum reinf area         = 1.5343E+00
```

- Footing two-way shear capacity (LRFD 5.13.3.6.3)

For sections without transverse reinforcement:

$V_n = [0.063 + 0.126 / \beta_c](f'_c)^{1/2} \times b_o d_v \leq 0.126(f'_c)^{1/2} \times b_o d_v$

b_o = Perimeter of the critical section at $0.5d_v$
β_c = Ratio of long to short side of load application = 1.0
$d_v = 0.9 \times 42 = 37.8$"

For column pin =>

P_f = 1443 kips (STR-IIA)

$b_o = 3.1416 \times (25.6 + 37.8) = 199\ in^2$
$\phi V_n = 0.9 \times 0.126(f'_c)^{1/2} \times b_o d_v = 1706$ kips > 1443 kips Okay

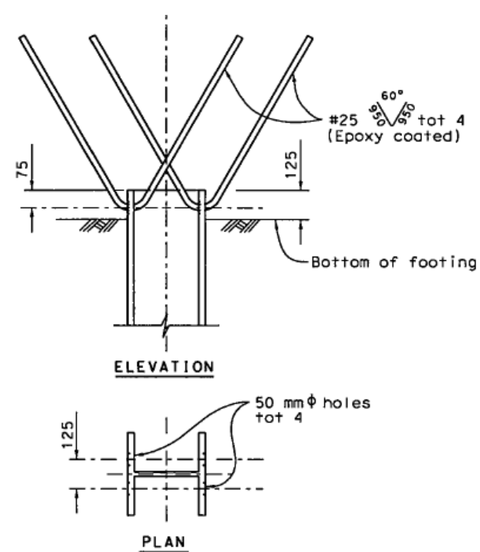
For pile =>

P_f = 251 kips (STR-IIA)

$b_o = 4 \times (14 + 37.8) = 207\ in^2$
$\phi V_n = 0.9 \times 0.126(f'_c)^{1/2} \times b_o d_v = 1775$ kips > 251 kips Okay

- Pile dowel embedment in footing is based on LRFD 5 and Caltrans Amendments

ϕ = 1.0 (Caltrans Amendments 5.5.5)



P_f = -180 kips (seismic)
Rebar tension capacity = 8 x 0.79 x 60 cos(30°) = 328 kips > 180 kips Okay
Rebar shear capacity = 4 x 0.8 x 0.48 x 80 x 0.79 x 2 = 194 > 180 Okay (LRFD 6.13.2.7)
Development $L_{db} > 1.25A_b f_y/(f'_c)^{1/2} = 1.25 \times 0.79 \times 60 / (4)^{1/2}$ = 30" < 37" Okay

5.5 PILES

- Several programs are available to perform lateral pile analysis/design as shown below.

Program	Description
LPILE	Lateral single pile analysis
GROUP	Lateral pile group analysis

It will be assumed that the geotechnical engineer has provided the vertical and lateral capability of the piles based on soil failure and movement

$P_{service}$ = 140 kips / -100 kips
$V_{service}$ = 10 kips

$P_{ultimate}$ = 280 kips/-200 kips
$V_{ultimate}$ = 30 kips

- Pile capacity for non-seismic loads is based on 10.5.5.2.3 with Caltrans Amendments 10.5.5.2.3

Axial compression:

Service: 140 kips > 133 kips Okay

Strength:

Reduction factor = 0.7 (Caltrans Amendments 10.5.5.2.3)
ϕP = 0.7 x 280 = 196 kips < 251 kips – No good (increase pile capacity or numbers)

Axial tension:

Reduction factor = 1.0 (Caltrans Amendments 5.5.5 for seismic)
ϕP = 1.0 x 200 = 200 kips > 180 kips Okay

Lateral:

Service: $((0.3)^2 + (6.7)^2)^{1/2}$ = 6.7 kips < 10 kips Okay

Strength:

Reduction factor = 1.0 (Caltrans Amendments 5.5.5 for seismic)
ϕV = 1.0 x 30 = 30 kips > 23 kips Okay

- Pile capacity for structural strength is okay by inspection

Structural strength does not usually control for steel H-piles

SECTION 6.0 DETAILING

The bent details are shown on the following sheets

Ⓐ	Sta 21+32.520 "HH" Line / Sta 145+17.000 "E5" Line	N 570437.601 / E 2063797.774
Ⓑ	Sta 19+38.604 "M" Line / PRC	N 570256.107 / E 2063769.228
Ⓒ	Sta 21+74.483 "M" Line / EC	N 570393.287 / E 2063772.565
Ⓓ	Sta 21+74.874 "M" Line / BC	N 570474.072 / E 2063818.521
Ⓔ	Sta 23+27.874 "M" Line / EC	N 570620.636 / E 2063857.755
Ⓕ	Sta 146+00.000 "E5" Line / POT	N 570488.410 / E 2063732.142
Ⓖ	Sta 144+00.000 "E5" Line / POT	N 570365.980 / E 2063890.290

Bench Mark and Control Point

Pt #1016 Elev 355.654
Found 2½" C.D.O.T. Brass Disk in well stamped "CDH 485+39.76 BC Mass. Nor = CL Mass" 208 meters south Highland Avenue.
N = 570564.954 E = 2063778.874

Pt #2007 Elev 355.577
Found 2½" C.D.O.T. Brass Disk in top curb stamped "CDOT CL 19th St - LT R/W Line PI" 25.91 meters east of intersection of 19th Street and Davidson Avenue.
N = 570357.175 E = 2063723.006

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL
8	SBd	215			

REGISTERED CIVIL ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER KOSALRAM KRISHNAN No. C53378 Exp. 6-30-07 CIVIL STATE OF CALIFORNIA

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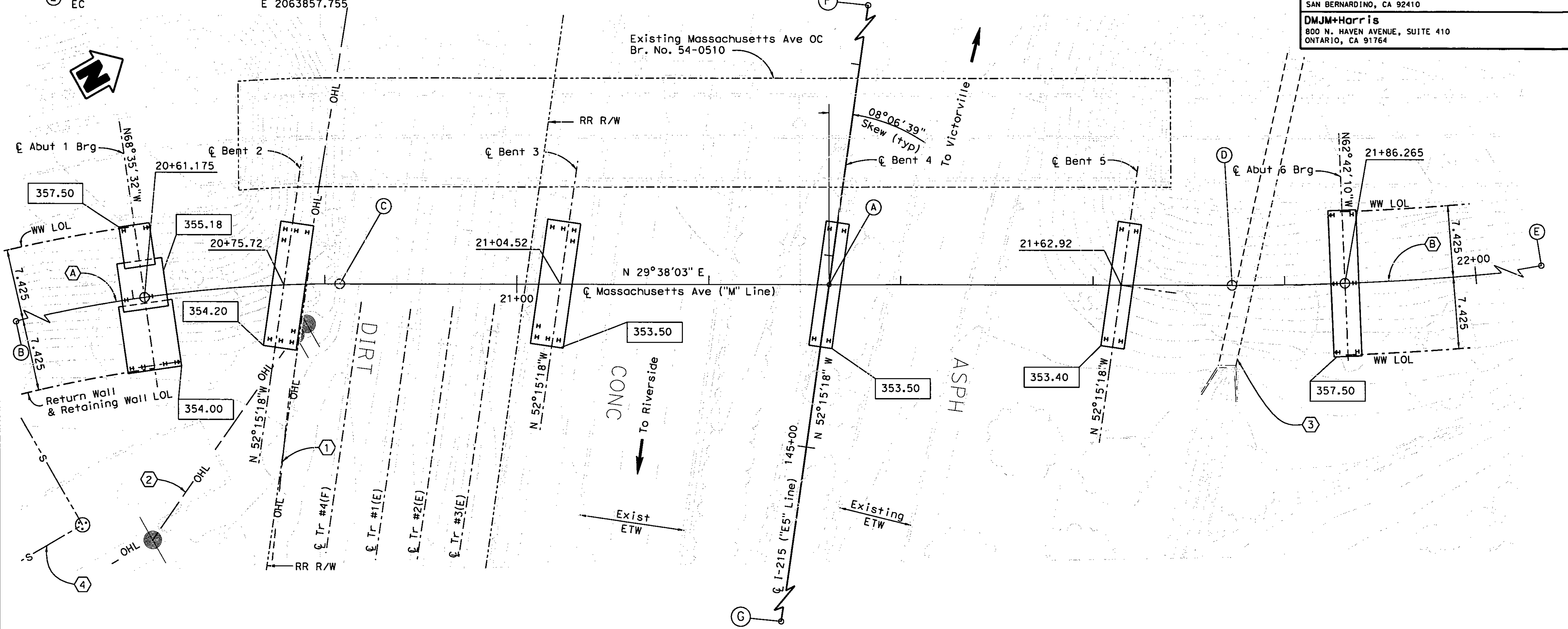
SAN BERNARDINO ASSOCIATED GOVERNMENTS
1170 WEST 3RD STREET
SAN BERNARDINO, CA 92410

DMJM+Harris
800 N. HAVEN AVENUE, SUITE 410
ONTARIO, CA 91764



UTILITIES

1. Telephone, overhead AT&SF (E) to be relocated by others
2. Electric, overhead SCE 12KV (E)
3. Existing RCB, Protect in place
4. Existing 200 mm Sewer (COSB)

CURVE DATA				
CURVE	R	△	L	T
Ⓐ	145.000	56°28'52"	142.939	77.881
Ⓑ	300.000	29°17'43"	153.391	78.411

PLAN
1:200

NOTES

- - - - - Existing (E)

353.00 Indicates bottom of footing elevation

All piles not shown for clarity

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

GEOTECHNICAL PROFESSIONAL APPROVAL DATE

DESIGN OVERSIGHT KAMMY BHALA	SCALE:	VERT.DATUM NGVD-1929	HORZ.DATUM NAD-83	DESIGN BY K. Krishnan	CHECKED
SIGN OFF DATE	PHOTOGRAMMETRY AS OF:		ALIGNMENT TIES	DETAILS BY E. Landas	CHECKED
	SURVEYED BY		DRAFTED BY	QUANTITIES BY	CHECKED
	FIELD CHECKED BY		CHECKED BY		

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

GREGORY V. BROWN
PROJECT ENGINEER

BRIDGE NO. 54-1265
KILOMETER POST 14.51

MASSACHUSETTS AVE OC (REPLACE)
FOUNDATION PLAN

FOUNDATION PLAN SHEET (METRIC) (REV. 3/1/99)

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 08-235
EA 007191

DISREGARD PRINTS BEARING EARLIER REVISION DATES 06-06-04 04-06-06

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET 5 OF 30

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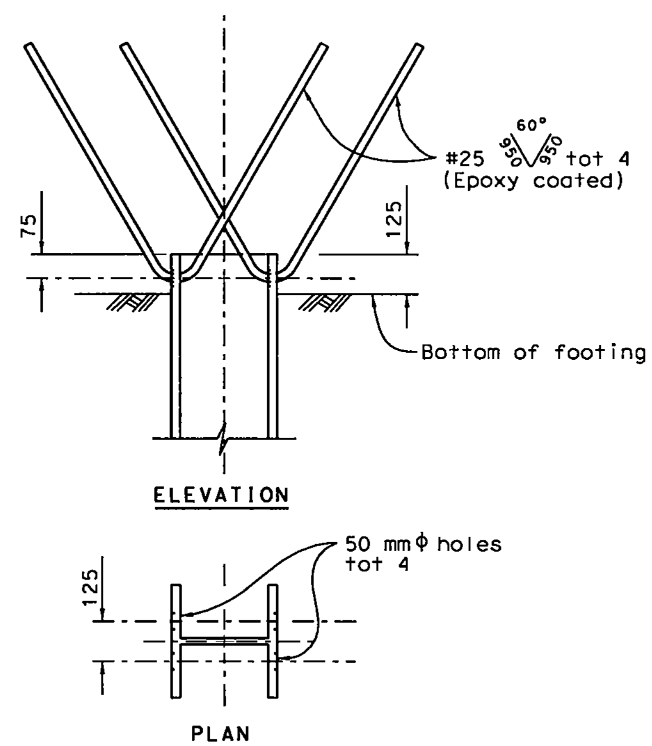


(HP 360 Pile)

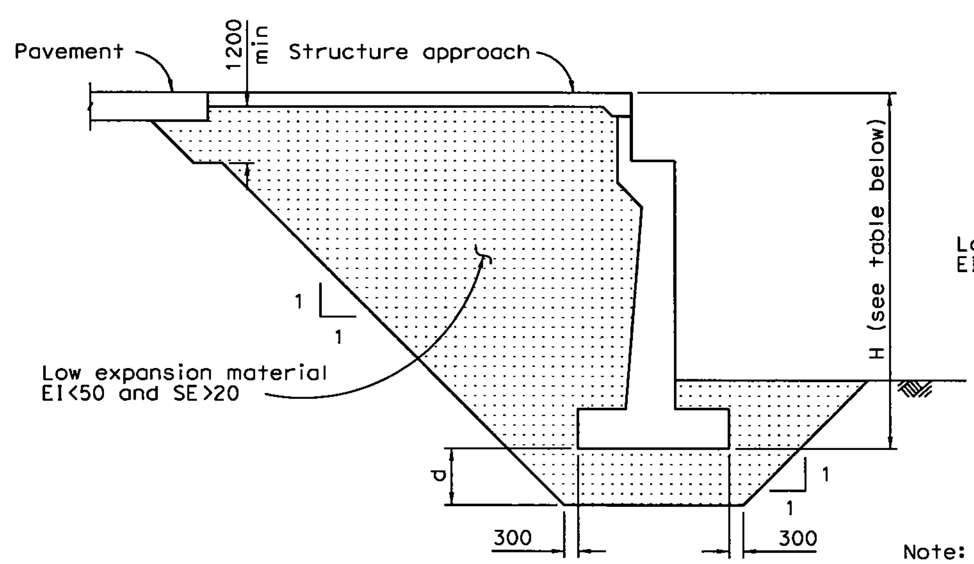
STEEL PILE ANCHOR

DESIGN CAPACITY:
Tension = 600 kN maximum
(Nominal axial reistance)

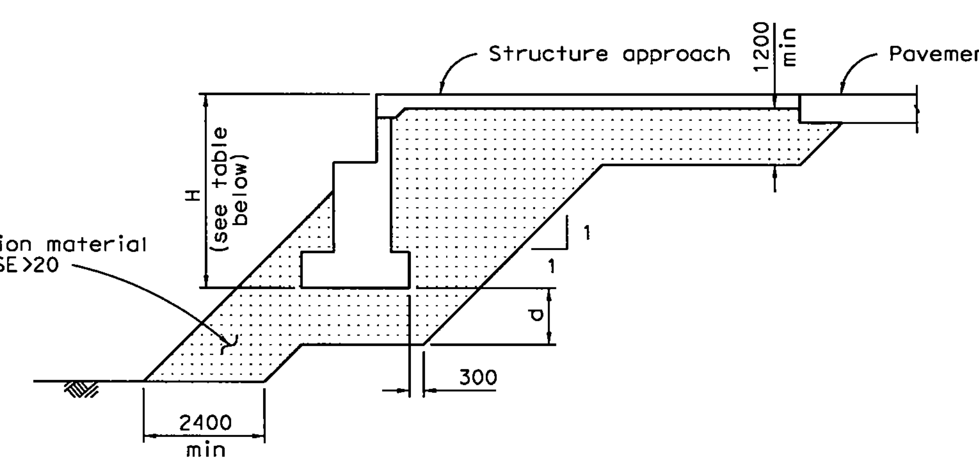
PILE DATA TABLE

LOCATION	PILE TYPE	DESIGN LOADING	NOMINAL RESISTANCE		DESIGN TIP ELEVATION	SPECIFIED TIP ELEVATION
			COMPRESSION	TENSION		
ABUT 1	HP 360 x 132	625 kN	1250 kN	600 kN	+340.0 (1), +340.5 (2), +344.0 (3)	+340.0
BENT 2	HP 360 x 132	625 kN	1250 kN	550 kN	+340.0 (1), +340.5 (2), +344.0 (3)	+340.0
BENT 3	HP 360 x 132	625 kN	1250 kN	550 kN	+340.0 (1), +340.5 (2), +344.0 (3)	+340.0
BENT 4	HP 360 x 132	625 kN	1250 kN	550 kN	+340.0 (1), +340.5 (2), +344.0 (3)	+340.0
BENT 5	HP 360 x 132	625 kN	1250 kN	550 kN	+340.0 (1), +340.5 (2), +344.0 (3)	+340.0
ABUT 6	HP 360 x 132	625 kN	1250 kN	600 kN	+342.0 (1), +342.5 (2), +348.0 (3)	+342.0

Design tip elevation is controlled by the following demands:
(1) Compression; (2) Tension; (3) Lateral Loads



ABUTMENT 1



ABUTMENT 6

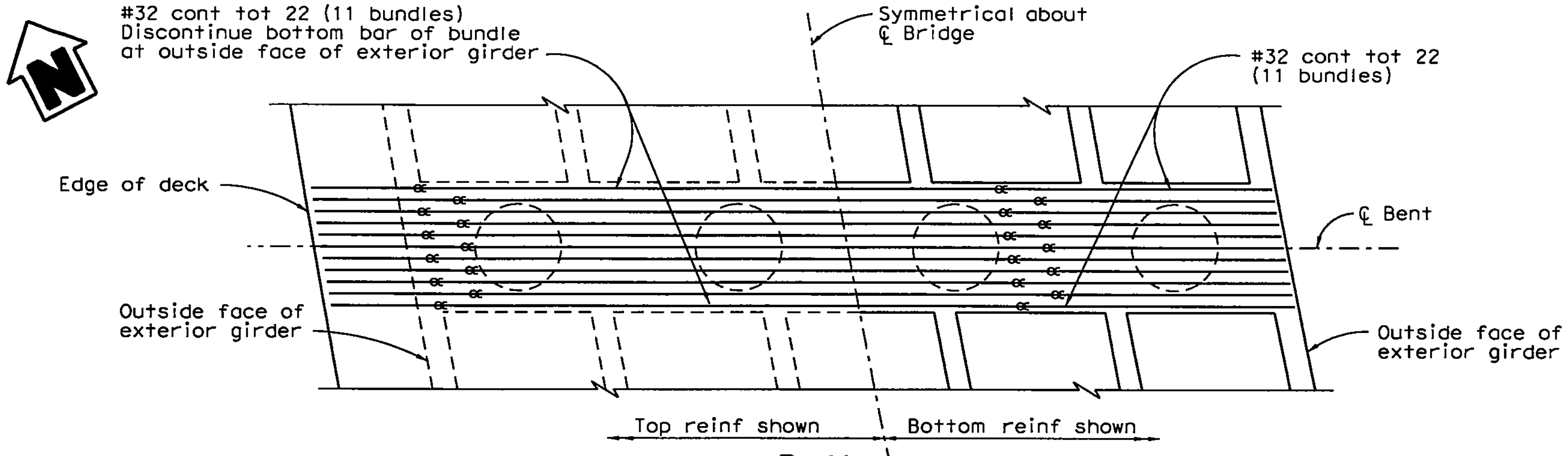
Note: Expansion index to be determined by ASTM D4829
Sand equivalent to be determined by California Test Method 217

H	d
<4.9 m	1.2 m
>4.9 m	0 m

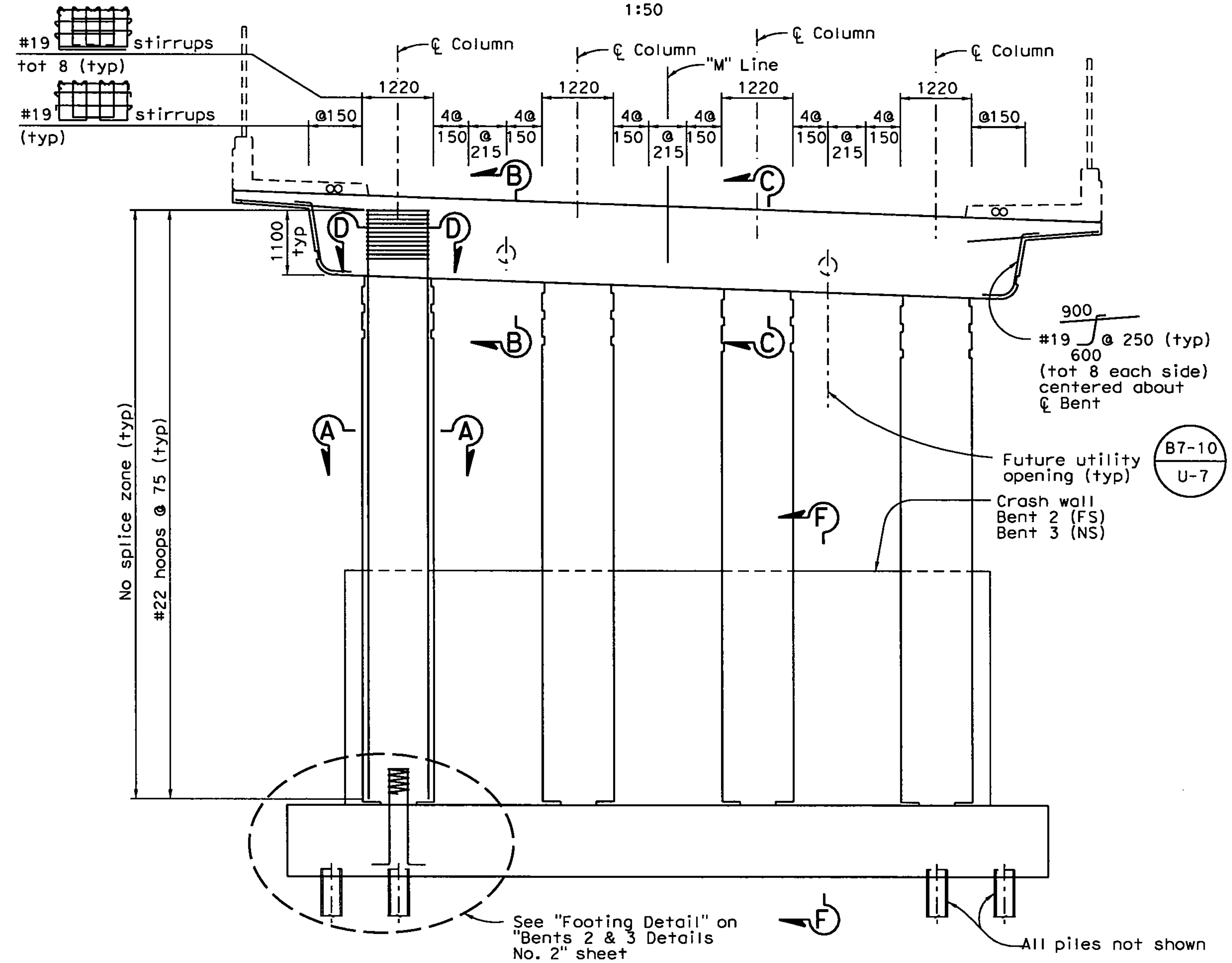
LOW EXPANSION MATERIAL IN BRIDGE EMBANKMENT

No Scale

ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SHOWN



PLAN
1:50



ELEVATION
1:50

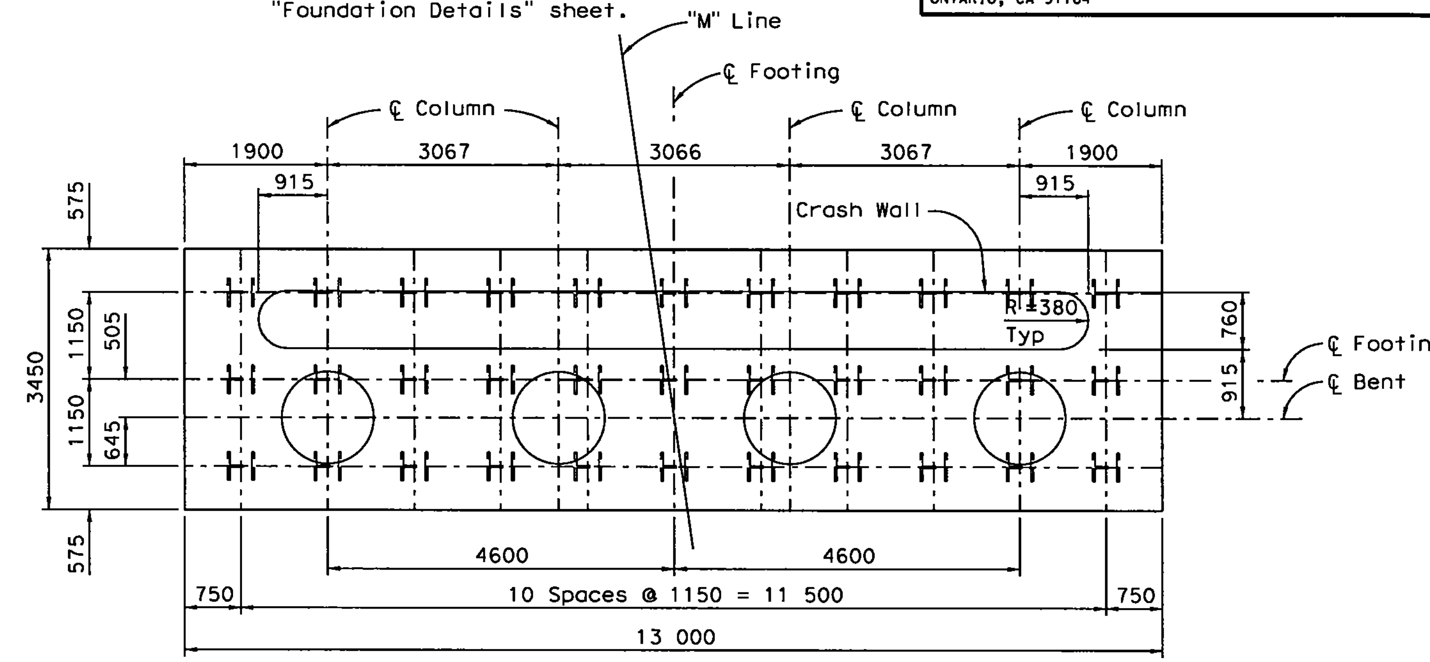
Legend:

H Indicates vertical HP steel pile

∝ Indicates bundled bars

Notes:

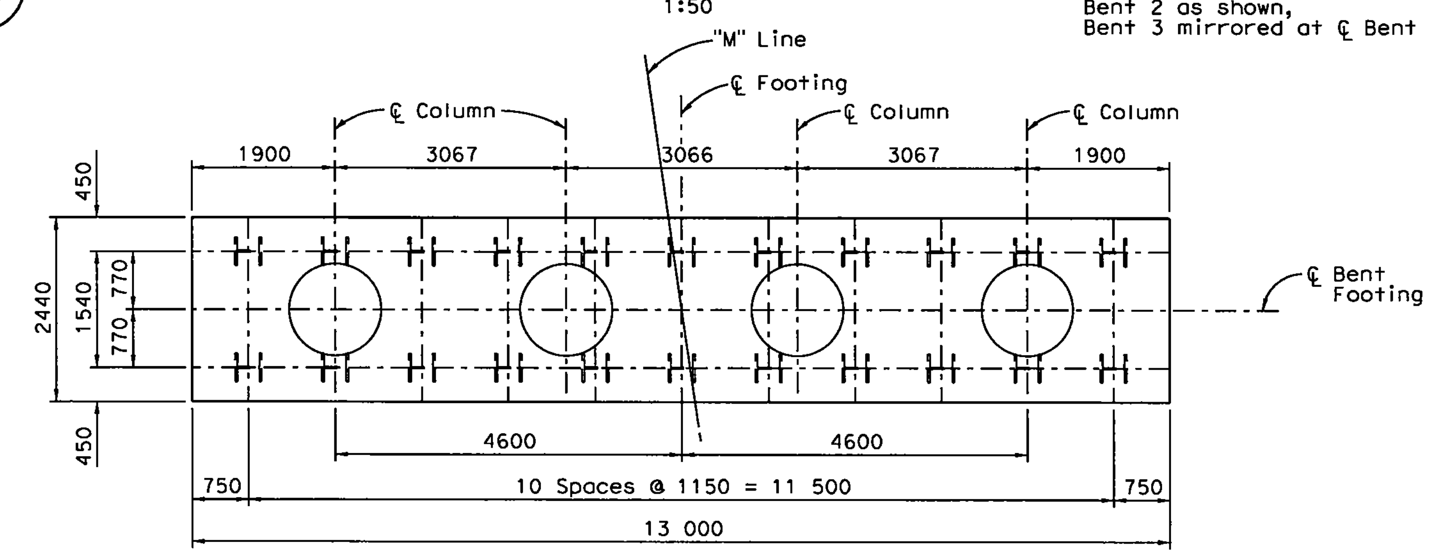
1. For Sections A-A, B-B & C-C, see "Bent Details No. 2" sheet.
2. For Bent Aesthetic Details and Sections D-D & F-F, see "Bent Details No. 3" sheet.
3. For Utility openings, see "Typical Section" sheet for location and size.
4. All piles shall be anchored per "Steel Pile Anchor" detail on "Foundation Details" sheet.



FOOTING PLAN - BENTS 2 & 3
1:50

Note:

Bent 2 as shown, Bent 3 mirrored at ℄ Bent



FOOTING PLAN - BENTS 4 & 5
1:50

ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SHOWN

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
8	SBd	215			

Caltrans Metric

REGISTERED CIVIL ENGINEER

PLANS APPROVAL DATE

KOSALRAM KRISHNAN No. C53378 Exp. 06-30-07 CIVIL STATE OF CALIFORNIA REGISTERED PROFESSIONAL ENGINEER

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SAN BERNARDINO ASSOCIATED GOVERNMENTS
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SAN BERNARDINO, CA 92410

DMJM+Harris
800 N. HAVEN AVENUE, SUITE 410
ONTARIO, CA 91764

DESIGN OVERSIGHT KAMMY BHALA

SIGN OFF DATE

DESIGN	BY J. Wong	CHECKED
DETAILS	BY E. Landas	CHECKED
QUANTITIES	BY	CHECKED

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

GREGORY V. BROWN
PROJECT ENGINEER

BRIDGE NO. 54-1265
KILOMETER POST 14.51

MASSACHUSETTS AVE OC (REPLACE)
BENT DETAILS NO. 1

DESIGN DETAIL SHEET (METRIC) (REV. 3/1/98)

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 08-235
EA 007191

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 04-06-06

SHEET 13 OF 30

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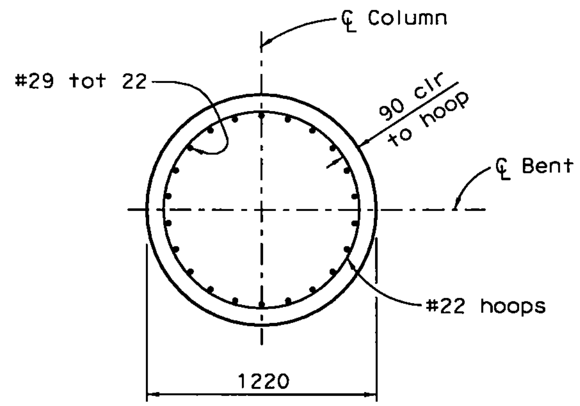
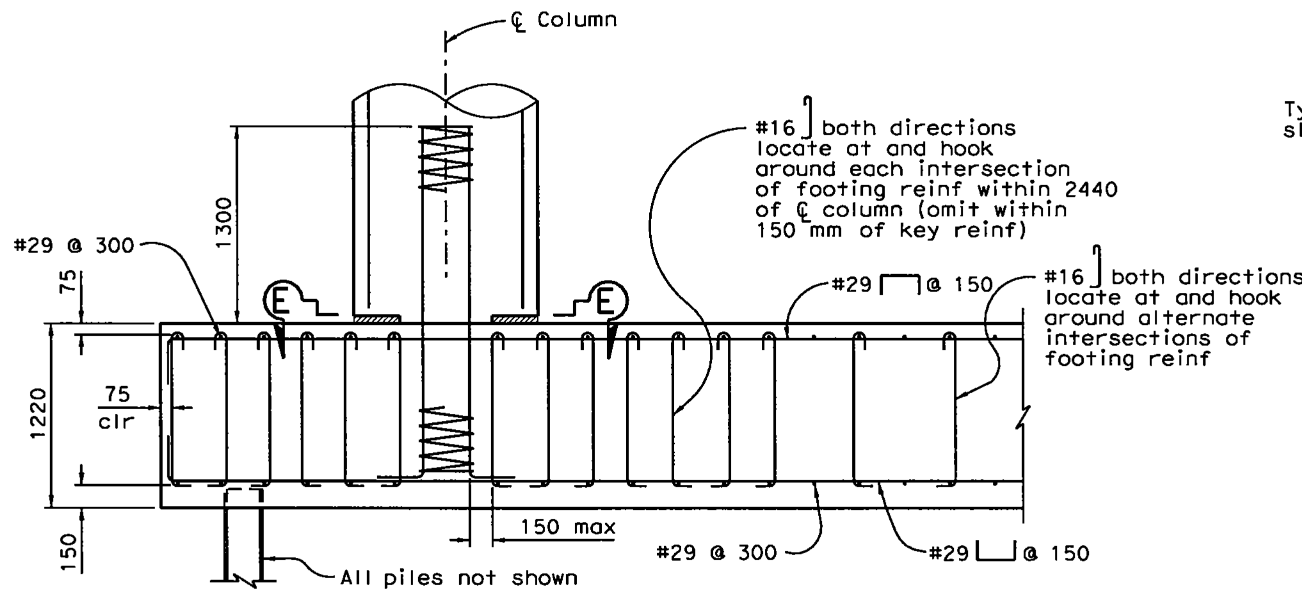
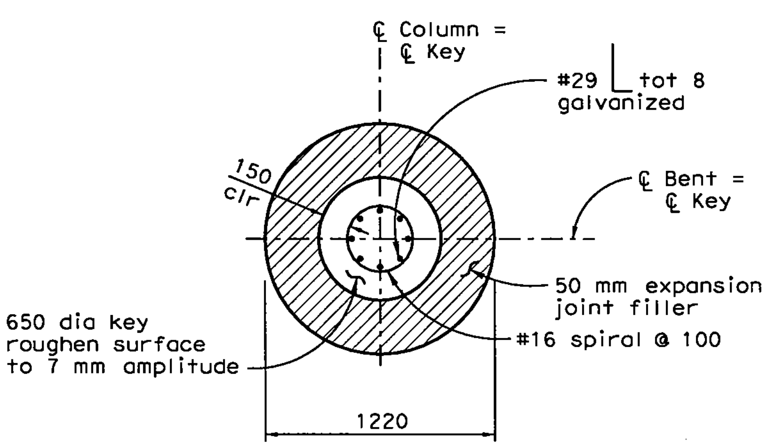


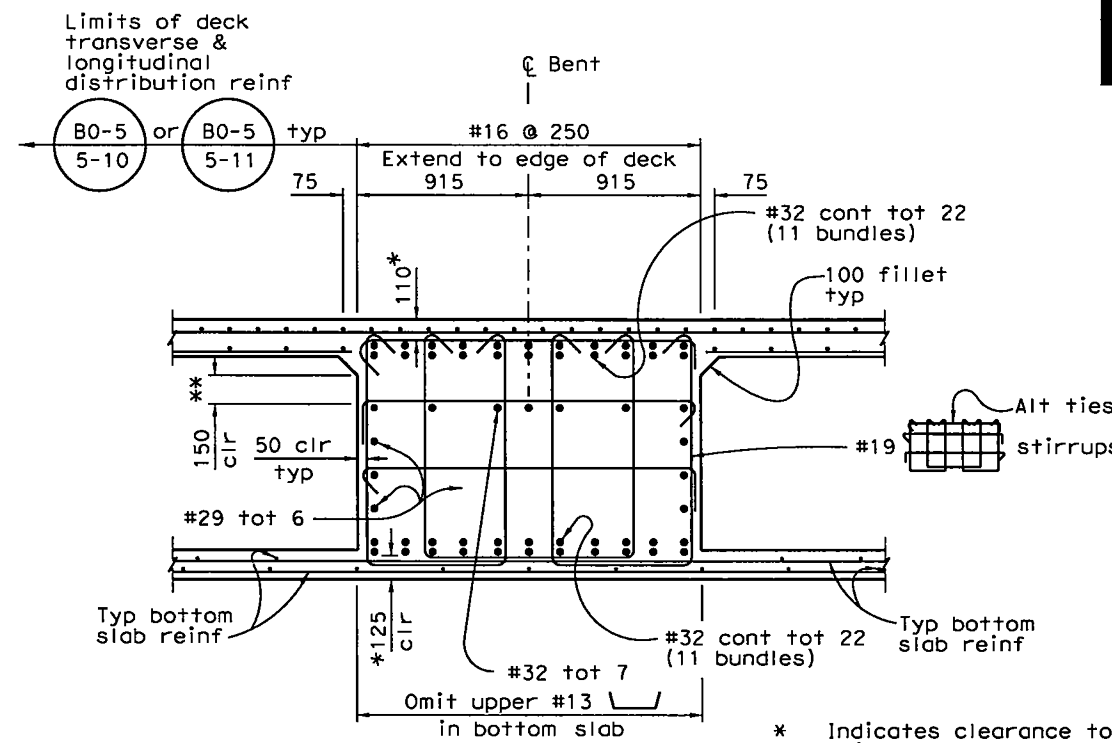
SECTION A-A
1:20



FOOTING DETAIL
1:25



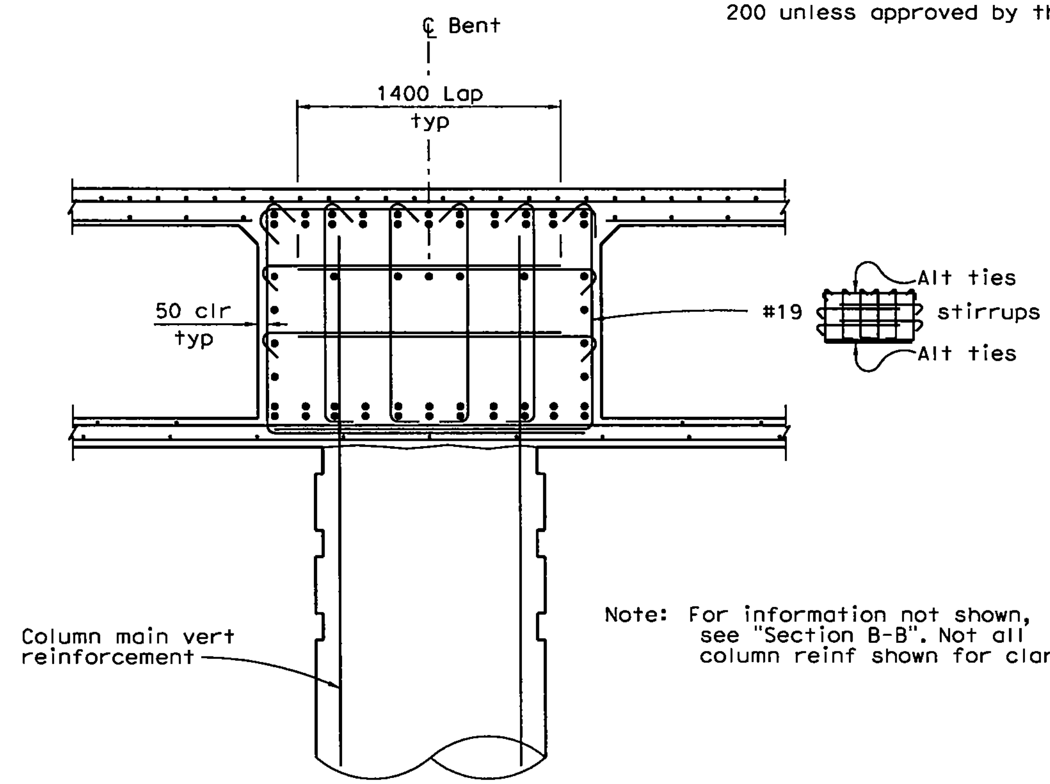
SECTION E-E
1:20



SECTION B-B
1:20

* Indicates clearance to main cap reinforcement

** Indicates clearance to #32 tot 7. Reinforcement may be lowered to clear prestress ducts; however, this dimension shall not exceed 200 unless approved by the Engineer.



SECTION C-C
1:20

Note: For information not shown, see "Section B-B". Not all column reinf shown for clarity.

ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SHOWN

Caltrans Metric

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
8	SBd	215			

REGISTERED CIVIL ENGINEER

REGISTERED PROFESSIONAL ENGINEER KOSALRAM KRISHNAN No. C53378 Exp. 06-30-07 CIVIL STATE OF CALIFORNIA

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DESIGN OVERSIGHT KAMMY BHALA	DESIGN	BY J. Wang	CHECKED	
SIGN OFF DATE	DETAILS	BY E. Landas	CHECKED	
	QUANTITIES	BY	CHECKED	

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

GREGORY V. BROWN
PROJECT ENGINEER

BRIDGE NO. 54-1265
KILOMETER POST 14.51

MASSACHUSETTS AVE OC (REPLACE)
BENT DETAILS NO. 2

DESIGN DETAIL SHEET (METRIC) (REV. 3/1/98)

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 08-235
EA 007191

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 04-06-06

SHEET 14 OF 30

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DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
8	SBd	215			

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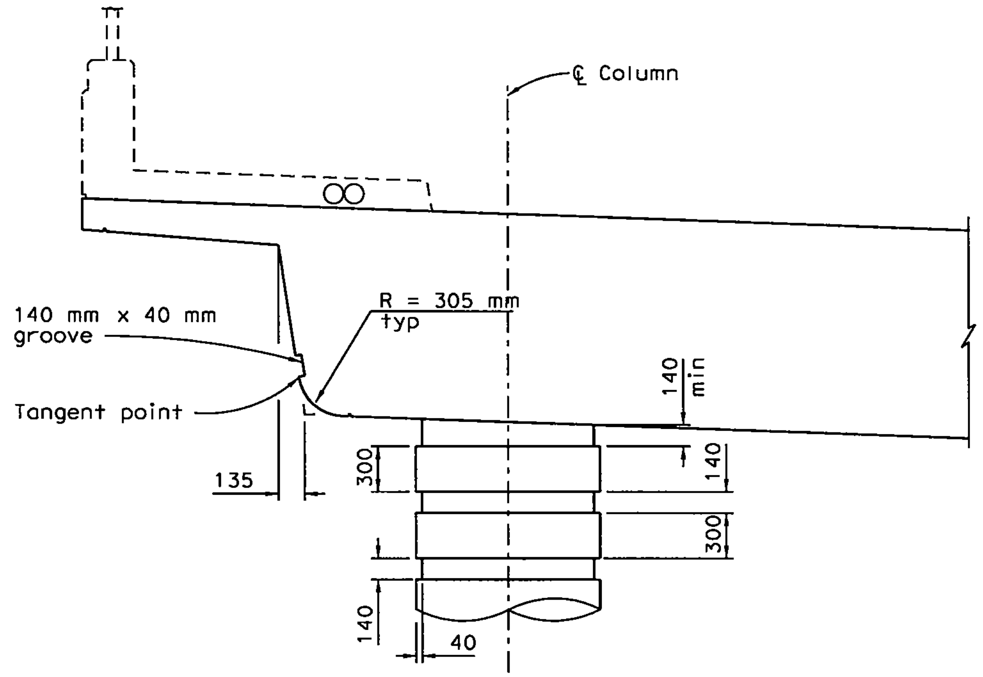
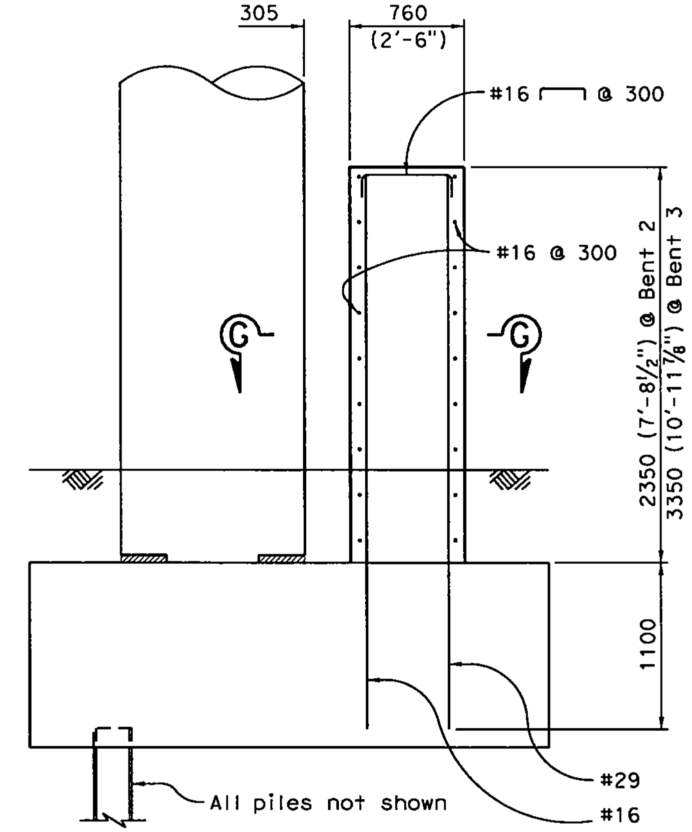


COLUMN/BENT-AESTHETIC DETAILS

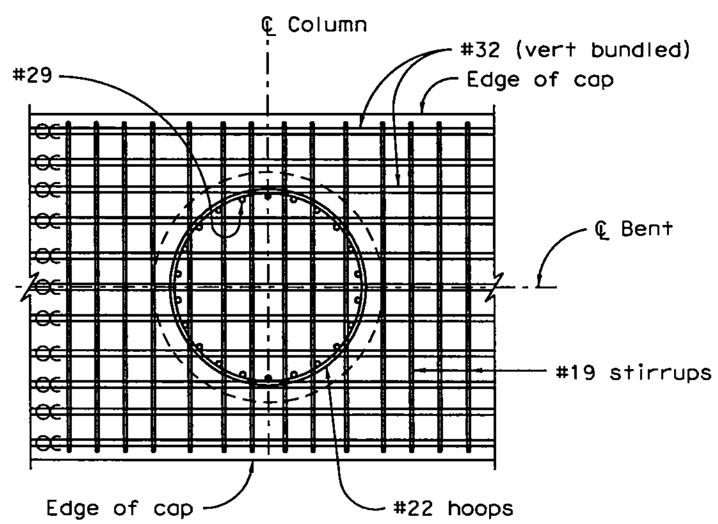
PART ELEVATION

1:25



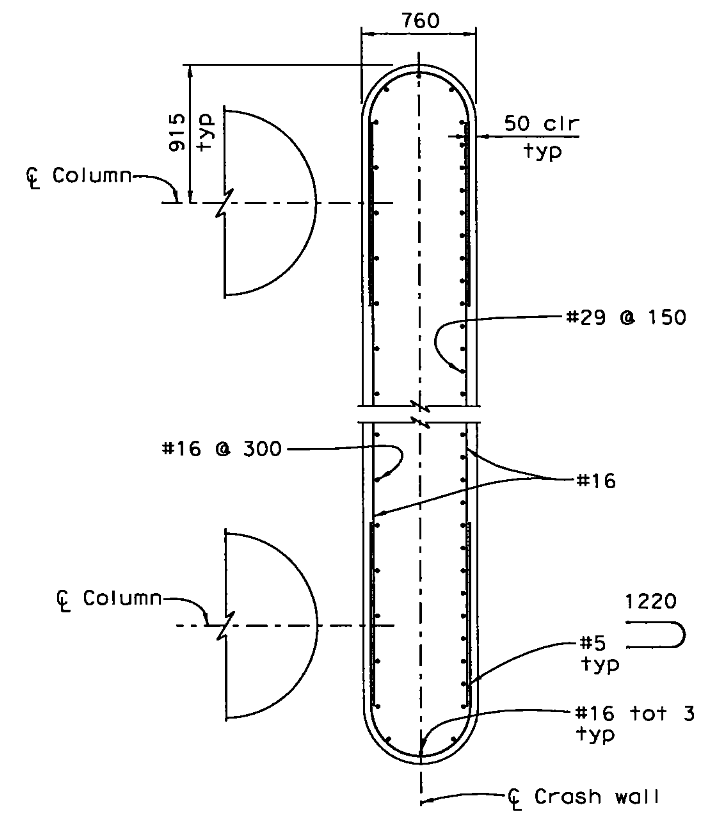
SECTION F-F

1:25



SECTION D-D

1:20



SECTION G-G

1:25

ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SHOWN

DESIGN OVERSIGHT KAMMY BHALA

SIGN OFF DATE

DESIGN	BY J. Wang	CHECKED	
DETAILS	BY L. Reyes	CHECKED	
QUANTITIES	BY	CHECKED	

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

GREGORY V. BROWN
PROJECT ENGINEER

BRIDGE NO. 54-1265

KILOMETER POST 14.51

MASSACHUSETTS AVE OC (REPLACE)

BENT DETAILS NO. 3

DESIGN DETAIL SHEET (METRIC) (REV. 3/1/98)

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 08-235
EA 007191

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REVISION DATES (PRELIMINARY STAGE ONLY) 04-06-06

SHEET 15 OF 30

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